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Prefabricated Vertical Drains and Pavement Drainage Systems

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Foreword

Four of the papers of this Record are on advances in pavement drainage systems and the other three are on prefabricated vertical drains. Both subjects are of interest to highway engineers.

Baldwin and Long describe the experience of the West Virginia Department of Highways with the concept of free-draining bases. They investigated a 4-in. bituminous stabilized open-graded base course connected to the surface drainage by aggregate-filled fabric underdrains. Design and construction are discussed, and results of evaluation of the pavement system are presented.

Highlands and Hoffman report on results of an evaluation of different types of subbases ranging from an impermeable cement-stabilized material to a very permeable, uniformly graded crushed aggregate. They also present data on falling weight deflectometer deflection measurements and roughness measurements.

Dempsey presents results of a study conducted to determine the core flow capacity requirements of geocomposite fin-drain materials used in pavements. He relates laboratory and field flow measurements to a standard pipe and sand envelope system and compares performance of pavement with fin-drain subdrainage with that of some standard systems.

Hinshaw compares the effectiveness of Hydraway edgedrain with that of the Ohio Department of Transportation's standard 4-in. pipe underdrain without fabric wrap. He indicates, on the basis of available data, that neither system has shown consistently superior performance over the other. Because there are gaps in the data, he plans to continue collecting data until definite conclusions can be reached.

Sarkar and Castelli report on a comparative study that involved wick drains and a preloading program and preloading without the use of wick drains to consolidate soft clay soils. They indicate that after a year an average of 80 percent consolidation of soil was achieved in the area with wick drains; less than 30 percent was observed in the nondrain area.

Kyfor et al. used prefabricated vertical drains to accelerate consolidation of soft foundation soils. They monitored the performance of the drains with piezometer and settlement-recording devices. An evaluation of the performance and effectiveness of the prefabricated vertical drain system in presented.

Saye et al. used wick drains to facilitate strength gains of foundation clays during stage construction and to preconsolidate the clays under a surcharge before paving. They report that wick drains allowed the excess pore water pressures generated by fill placement to dissipate. They observed significant differences in the apparent horizontal coefficient of consolidation due to variations in wick drain spacing.

Design, Construction, and Evaluation of West Virginia's First Free-Draining Pavement System

JOHN S. BALDWIN AND DONALD C. LONG

The adverse effects of entrapped water within the various components of a pavement system are generally accepted and well-known. In an effort to reduce, if not eliminate, the detriment of the entrapped water, the West Virginia Department of Highways constructed its first project using the free-draining concept in the summer of 1982. The project chosen for experimentation involved over 5 mi of four-lane divided highway. Incorporated in this full-depth asphaltic pavement is a 4-in. bituminous, stabilized, open-graded base course which is connected to the surface drainage by numerous aggregate-filled fabric underdrains. This paper discusses the design considerations given this project, the construction sequence used, and the evaluation of the pavement system after 4.5 yr of service.

It is now generally accepted in West Virginia, as in many other states, that excessive and uncontrolled water entrapped within the components of a traditionally designed pavement system will eventually lead to pavement distress. In an effort to reduce, if not eliminate, the detriment of the entrapped water, the West Virginia Department of Highways constructed its first project using the free-draining approach during the summer of 1982. The project was on an Appalachian Development Corridor and involved a 5.2-mi section of four-lane divided highway.

The project chosen for experimentation had been designed for a 10-in. portland cement concrete pavement over a dense 6-in. crushed aggregate base course which in turn overlaid a dense 6-in. granular subgrade.

Construction had been completed to the top of the subgrade layer when it was decided to redesign the project to permit bidding on equal alternate pavement systems consisting of either full-depth asphaltic concrete or portland cement concrete. Both of the alternate systems were to incorporate a free-draining layer.

DESIGN

When consideration was given to using a free-draining layer, the first question to arise involved the gradation of the aggregate. It seemed apparent that a reasonable balance between drainability and stability would be necessary because optimum permeability and good stability tend to be opposing factors. If consideration had to be given only to the drainage aspect, round particles of a single size would appear to be the best suited, but there would be little strength or stability in such a material. Conversely, a very dense and stable material would most likely lead to poor drainage characteristics. Based on previous but limited experience, an AASHTO 57 gradation appeared to be well suited, especially because a standard gradation was desirable. That particular gradation exhibited a satisfactory interconnected void space while at the same time appeared to have reasonably good interlock between particles for stability. In order to enhance the stability, especially during the construction phase, it was decided to add 2 percent bituminous material to the aggregate, which was expected to be just enough to completely coat each individual particle and bind the mass together.

Where a free-draining layer should be placed in a pavement structure is a subject of some debate. There appear to be legitimate and compelling reasons for locating the layer at the bottom of the pavement structure as well as at the top or somewhere in between (*1*). Weighing the advantages and disadvantages of each pavement configuration, it was decided to place the free-draining layer as a base course at the bottom of the pavement structure in the hope that, by doing so, a greater thickness of the pavement structure could be kept free from water accumulation.

Ideally, the thickness of a free-draining layer should be based on the amount of expected inflow into the pavement system. Calculating that inflow, however, proved to be a difficult task because West Virginia's conventional pavements are designed to keep as much water out of the pavement system as possible. Based on engineering judgment and the moderate levels of rainfall encountered throughout the state, a 4-in.-thick free-draining layer was believed to be adequate.

There appeared to be two options possible for removing the water from the free-draining layer: either concentrate the drainage and channel it into controlled outlet points or "daylight" the free-draining layer into the surface drainage. Although both concepts have been used in West Virginia since the first free-draining project was constructed, the initial project used the controlled outlet approach. In this project, water is directed toward a "V" ditch which was constructed parallel to the alignment of the roadway underneath the shoulder. At 100-ft intervals, aggregate filled underdrains intersect the "V" ditch, thus allowing the water an easy flow path to the surface drainage.

West Virginia Department of Highways, Materials Control, Soils and Testing Division, 312 Michigan Avenue, Charleston, W. Va. 25311.

The only other aspect of the free-draining layer that needed to be addressed before commencing such a project was how to protect the free-draining base from clogging. The gradation of the granular subgrade was such that standard filter calculations indicated the need for a filter between the subgrade and the free-draining base. Again, alternates were permitted in the contract. Either an appropriate geotextile filter or a 2-in. layer of filter aggregate was permissible.

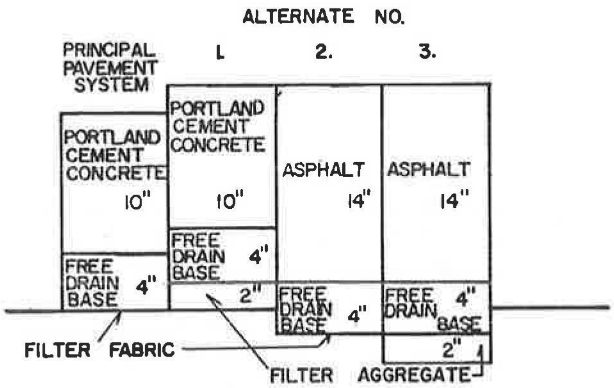

FIGURE 1 Alternate free-draining pavement designs.

The project was advertised with four alternates (Figure 1)—two for the portland cement concrete pavement and two for the full-depth asphalt. The successful contractor chose the full-depth asphalt with the filter fabric alternate. In cross section it appeared as shown in Figure 2. As is evident in the drawing, the fabric wrapped over the top of the free-draining base to protect against the infiltration of fines from the overlying shoulder stone.

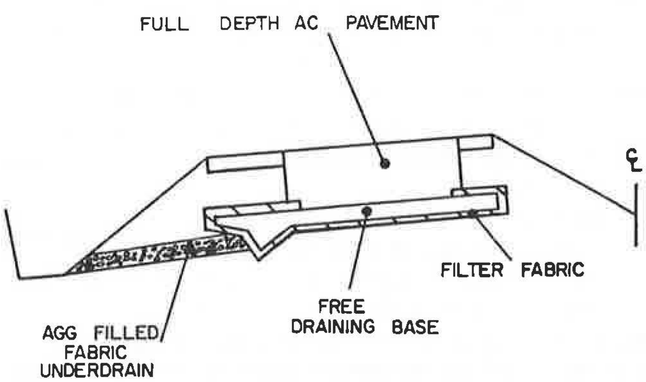

FIGURE 2 Free-draining pavement in cross section.

CONSTRUCTION

Work was begun on the project on June 30, 1982. The first order of business was to shape the existing subgrade to template. That required removal of approximately 2 in. of the existing granular subgrade and included the excavation of the lateral "V" ditch.

Next, the contractor installed the aggregate-filled fabric underdrains. He cut a narrow trench into the subgrade of the shoulder at a 45 degree downgrade angle to the roadway alignment. The trenches were then lined with engineering fabric and backfilled with uncrushed pea gravel as shown in

FIGURE 3 Cutting of the underdrain trenches.

FIGURE 4 Installation of the aggregate-filled fabric underdrains.

Figures 3 and 4. Underdrains were generally constructed at 100-ft intervals.

The next phase was to place the geotextile filter over the subgrade as shown in Figure 5. The geotextile filter covered the entire main alignment of the roadway and extended far enough laterally to completely line the lateral "V" ditch. Where the aggregate-filled fabric underdrains intersected the "V" ditch, a piece was cut from the fabric in the ditch. The fabric from the underdrain was then pulled through and laid open to provide direct contact between the pea gravel and the free-draining material which was to be subsequently placed.

As shown in Figure 6, the "V" ditch was the first area to receive the free-draining base. This had been a matter of some concern. Some contractors had expressed doubt that reasonable stability could be achieved where the material was placed this thick (up to 16 in.). It was placed in a single lift with a side paver. The placement occurred essentially without incident and instability was never a problem.

FIGURE 5 Geotextile filter placement.

FIGURE 6 Filling of the "V" ditch with free-draining base.

Once the "V" ditch had been filled and compacted enough to seat the stone, placement and rolling of the 4 in. of free-draining base on the main alignment of the roadway was accomplished. This was also done without any significant problems, although maneuvering the paving machine and trucks directly on top of the geotextile occasionally resulted in some tearing of the fabric which had to be patched.

As was found from working with free-draining bases on subsequent projects, a large part of a successful installation is based on allowing the placed material to cool to approximately 130°F to 150°F before rolling is attempted.

Subsequent operations that involved placing additional lifts of full-depth asphaltic pavement (for a total of 14 additional in.) also proceeded smoothly with the free-draining base apparently providing ample support. Figure 7 shows a cross section of the pavement structure under construction.

PERFORMANCE AND EVALUATION

In order to judge the effectiveness of any free-draining pavement system, it is apparent that four factors would have to be determined over a period of time:

- Amount of inflow into the pavement system,
- Amount of outflow from the pavement system,
- Amount of time necessary to rid the pavement system of intrusive water, and
- Durability of the pavement as compared to conventionally designed pavements.

FIGURE 7 Cross section of pavement under construction.

Although there was definite interest in how well this project performed as a whole, it was obvious from the length of this roadway (5.2 mi) that specific test sections would have to be defined in order to meet the objectives of the performance evaluation. Consequently, four test sections were chosen for experimentation and close monitoring.

Rather than base the evaluation of this free-draining system on obscure calculations, it was decided to attempt to periodically control the input of water into the system via borings in the pavement in each of the test sections and measure outflow versus time. It was also decided that perhaps the best way to determine whether the system was performing as intended was to install cased observation borings into the pavement structure, especially at the bottom of vertical curves. With the project equipped with an automatic rain gauge to quantify rainfall events, plans were made to periodically uncap these observation wells during or immediately after inclement periods to check for water buildup within the system.

In order to get an overview of the effectiveness of the entire project, comprehensive drainage adequacy surveys, coupled with informal pavement condition surveys, were planned to take place annually. These surveys were to provide information concerning the condition or adequacy of each individual drain in the experimental system as well as to provide an indication of the overall appearance and rideability of the pavement.

Water Injection Tests

Throughout the 3-yr evaluation period, controlled input and outflow measurements were made at each of the four pavement sections chosen for monitoring. Basically, the tests consisted of introducing metered water from a 400-gal water vessel, at a measured flow rate, into the 4-in.-diameter injection wells which extended vertically to the bottom of the free-draining base. All injection wells were located so that the water had to drain through both traffic lanes before exiting through the drainage outlets. The amount of time required to obtain flow from the applicable drain outlets was measured as were the outflow volumes which were experienced through the duration of the tests. Input into the system was generally discontinued when a constant outflow was achieved, but timed outflow measurements were continued until outflow essentially stopped.

A variety of input, outflow, and time measurements were recorded at each of the test sections over the 3-yr monitoring period. The primary influence on how fast water could purposely be introduced into the system appeared to be based on the maximum outflow capability of the vessel used to transport water to the project (6 gal/min) and on the amount of particulate matter (road dust, fine cinders, etc.) that fell into the wells from around the embedded well caps each time they were opened for testing or observation. In the beginning, newly drilled injection wells appeared to easily take on the maximum 6 gal of water per min that was available for input. In fact, there were indications that a much greater timed volume could have been introduced if it were available. In time, however, with the cumulative addition of particulate matter, a substantial reduction in the amount of water that could be introduced into those same wells was experienced.

The particulate contamination appeared to restrict the flow only in the well, however, because replacement wells, installed adjacent to those exhibiting reduced flow, revealed no problem in accepting the maximum 6 gal/min input.

Measurement of the outflow from the system revealed that as little as 9 percent to as much as 100 percent of the water purposely introduced was recovered, but this should not be taken as an accurate indication of efficiency. In most cases, those measurements were heavily influenced by how effective the attempts were in inducing positive flow from the actual outlets in the test sections. Not only were the aggregate filled underdrains capable of passing water into the soil at virtually any point along their length, but the biggest problem involved accurately measuring the outflow from them. Initial attempts to raise the fabric outlet high enough to measure the outflow failed because the water simply flowed through the permeable fabric liner at the first point where the fabric left the soil. Pieces of "U" shaped steel guard rail were driven back under the drain outlets in an attempt to provide a more positive impermeable channel through which the water could flow.

Visual observations of flow activity at the outlets as well as from within the pavement and "V" ditch observation wells indicated that essentially all the water introduced directly into the pavement system had passed completely through the system (including the appropriate outlets) in no more than 4 hr from when the water was injected. The time necessary to rid the actual pavement structure of the water was undoubtedly less.

Inclement Weather Observations

One of the primary objectives of the evaluation was to observe the project during or shortly after rainfall events. Throughout the course of the 3-yr monitoring program, those objectives were met on numerous occasions, but because of the magnitude of the project, the majority of those observations were confined to the observation wells and underdrain outlets associated with the four test sections previously described.

According to rain gauge data, the performance of the project was observed during inclement periods that yielded as little as 0.11 in. of rain in 5 hr to as much as 1.79 in. in 20 hr. Individual underdrain outflow measurements made during these events indicated a range in flow rate from as little as one drop per sec to as much as 3.75 gal/min.

Of all the inclement weather observations made, the earliest and perhaps most dramatic observation occurred on May 4, 1983. Although it was not raining at the time the project was inspected, rain gauge charts indicated that 1.79 in. of steady rainfall had fallen on the project in the previous 20-hr period. Arrival at the project was at 9:00 a.m. and, according to the rain gauge data, the storm had slacked off and ended at around 5:00 a.m. that morning.

Observed outflow from all underdrains observed was impressive. In order to quantify the visual observations, measurements of outflow were made at the four defined test sections. Flow rates from 1.25 gal/min to as much as 3.75 gal/min were recorded within 1 hr after arrival (5 hr after rainfall had essentially ended).

Immediately after each test section outflow measurement was made, the associated wells in the free-draining base underlying the pavement, as well as those installed in the "V" ditch, were uncapped. Although all observation wells into the free-draining base were moist, no measurable water was observed. This indicated that the water had at least exited the main pavement structure. Conversely, measurable water (up to 7 in.) was found in some of the "V" ditch observation wells which decreased throughout the day.

Observations made on October 20, 1983, indicated that neither a large quantity of rainfall nor a buildup of measurable water within the "V" ditch was necessary before outflow occurred. Measurements of outflow collected that day revealed that flow rates as high as 0.18 gal/min were possible with only 0.11 in. of rainfall. Rain gauge data indicated that the 0.11 in. had accumulated in 5 hr, before the outflow measurement, and that no precipitation had fallen in the previous 24-hr period. While outflow rates increased as precipitation continued, measurable water was never found in any of the observation wells even though 0.44 in. of rain had fallen when the observations were discontinued.

Although the minimum amount of rainfall necessary to induce outflow has not been determined, it is believed that such quantification would be meaningless because of the numerous interrelated factors which would undoubtedly have a bearing on such measurements. Some of those factors include rainfall intensity, rainfall duration, temperature, humidity, the amount of previous rainfall, and the permeability or porosity of the pavement components.

In addition to obtaining actual outflow measurements, an attempt was made to visually inspect some of the other underdrains in the system. The inspection of those outlets that were found generally revealed good outflow. Many of the actual outlets, however, could not be positively located; but overly saturated soil at 100-ft intervals, adjacent to the shoulder, was often apparent. This was indicative that water was indeed exiting the system at most of the outlet points as intended.

Field Permeability Testing

It was obvious from outflow measurements collected during inclement periods that substantial volumes of water were infiltrating into the pavement system. While available literature indicates that water can infiltrate a pavement system from under the pavement or laterally from the sides or shoulders of a pavement, popular consensus is that the primary source of intrusive water that enters a pavement is from the pavement surface (2). Although this consensus is undoubtedly based on the assumption that most pavements contain cracks and joints, the permeability of the paving materials also appears to be taken into consideration. While this pavement had no joints or obvious cracks, an attempt was made to quantify water infiltration into the pavement system through permeability measurements of the pavement surface.

The device used to make the permeability measurements was an outflow meter. Although this device was originally designed to determine the macro texture of a pavement surface, the theory of operation was very similar to falling head permeability devices used in the laboratory.

The outflow meter is a Plexiglas™ cylinder, sealed at one end by a removable plunger. Attached to the inside of the cylinder at precise intervals are three electrodes that are interconnected to a digital stopwatch. The electrodes activate the stopwatch and stop it with the passing of a predetermined volume of water, thus making it possible to accurately determine the outflow from the device after raising the plunger. Assuming a watertight seal could be made and maintained between the circumference of the base of the unit and the surface of the pavement, the rate of outflow from the device would be directly proportional to the rate of inflow into the pavement.

During the construction of this project the outflow meter was used in conjunction with the free-draining base. That testing indicated that the free-draining layer was effective in passing at least 27 gal of water per square foot of surface per min. Attempts to obtain permeability values for the wearing surface indicated that material was essentially nonpermeable. Permeability testing of the asphaltic shoulder material was also attempted but was not successful because of failure to get a watertight seal between the base of the device and the shoulder material itself.

Assuming that the technique used in making the permeability determinations was valid, at least from the comparative standpoint, the pavement surface in this particular project is not believed to be the primary source for water that infiltrates into the pavement structure. The absorption of water by the shoulder material observed during this and other testing has led to the suspicion that the shoulder areas might be the primary source of infiltration.

Drainage Surveys

One of the priorities in evaluating the effectiveness of this free-draining system was to formally survey the drainage performance of each of the underdrain outlets. This was accomplished in the spring of each of the 3 yr of the monitoring period and turned out to be a much more arduous task than originally envisioned. Because of a successful seeding operation and because the length of the underdrains varied considerably, it was increasingly difficult to locate the individual drain outlets even though they were constructed on 100-ft centers. In addition, once drain outlets were located, it was difficult to evaluate their individual effectiveness uniformly because the performance level of the drains was not the same. This may have been caused, in part, by the intermittent precipitation rates that occurred during the survey periods or by the amount of water passing through the system that was available for outletting from one location to another.

In order to simplify the evaluation process, the following categories of drain adequacy were defined and used:

- Open and flowing or shows evidence of flow,
- Open and moist,
- Open,
- Covered but flowing or shows evidence of flow,
- Covered but moist, and
- Completely covered, blocked, or too low for effective drainage.

Without knowing anything else about the individual drains other than the condition of the outlets themselves, the first four categories were thought to indicate satisfactory performance, even though direct observation of flow was not always discernible. The condition associated with the last two categories was believed detrimental enough that drainage could not be considered adequate, at least for the rapid removal of water.

The summary of the drainage for the entire project is graphically shown in Figure 8. The bars depicted in the graph represent the drain adequacy for all drain outlets as surveyed in 1983, 1984, and 1985. Each total bar represents the percentage of the outlets actually found throughout the 5-mi project in each of the surveys. As shown, more of the drain outlets (60 percent) were found in the first year of the survey than in either of the subsequent survey years. The shaded portion of each bar represents the percentage of all the outlets in the project that

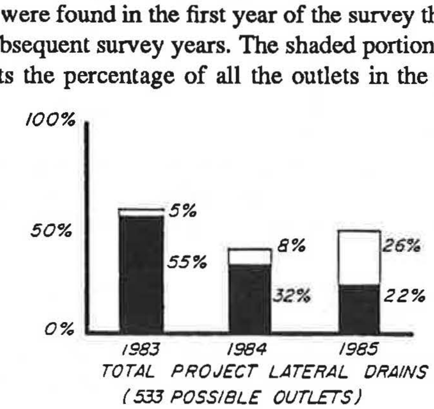

FIGURE 8 Drainage survey summary.

were found and judged to be capable of rapidly removing water from the pavement system. The unshaded area represents the percentage of outlets judged not to be performing satisfactorily. Overall, the surveys indicate a trend of decreasing effectiveness for rapidly outletting the water.

Pavement Appearance

Although the overall general appearance and rideability of this pavement is still relatively good after 4.5 yr of service, a gradual but noticeable decline in both pavement properties has been observed. This decline is attributed to two basic pavement problems, neither of which has been found to be related to the performance of the free-draining layer.

The first surface anomaly noticed affected mostly the rideability of the pavement. It involved the early occurrence of several transverse depressions which were apparent throughout the project. These depressions were primarily the result of differential settlement of the poorly compacted backfill associated with underlying transverse pipe.

The most noticeable surface anomaly that affected the appearance of the pavement was the occurrence of longitudinal cracks in the surface of the pavement. These cracks were prominent in several areas of the project and were mostly confined to the wheel paths of the outside lane as is shown in Figure 9.

FIGURE 9 Longitudinal surface cracking.

In 1986, several of the areas exhibiting the longitudinal cracks were core drilled. Examination of the cores indicated that the fractures generally extended only through the top inch of the wearing surface. The probable cause of the fractures has been attributed to an excessively hot liquid asphalt which was used in the batching of the wearing course. Consequently, this resulted in a thin coating of the aggregate particles and a relatively brittle mix.

Only slight rutting of the pavement surface has been observed. The wheel track rut depth has been measured on more than one occasion and was found to be comparable to other bituminous pavements of similar age and traffic.

CONCLUSIONS

The concept of using a drainage layer in pavement systems appears to be a worthwhile venture. Assuming that the effect of free water in conventional pavements is the major problem that it is widely reported to be, the 4.5 yr experience with West Virginia's first free-draining project would indicate that the use of a drainage layer can be effective in ridding pavement components of water buildup. Even though measurable free water has been documented in the subpavement collection trenches (located at the edge of the pavement), water has never been found to accumulate in the free-draining base within the actual pavement structure itself.

The most effective method of outletting the water from the pavement is still undecided. West Virginia's original free-draining project, which used a subpavement "V" shaped collection ditch coupled with aggregate filled engineering fabric underdrains, is continuing to be effective, but the increasing number of underdrain outlets that appear to be becoming clogged or blocked on that project is a source of concern. Even though the number of apparently inoperable drain outlets is now substantial, more underdrains were installed on this project than are probably necessary. Also, when considering those outlets deemed nonadequate, it should be kept in mind that water can and undoubtedly does pass through—even if not as rapidly and freely as would be desired.

Although it is apparent that West Virginia's first free-draining pavement is performing as intended, it is concluded from the experience gained from monitoring this project that the continued effectiveness could have been better ensured if a more positive and protected means of outletting water from the system had been employed. Recently constructed drainage projects in West Virginia have used corrugated plastic pipe with a protective concrete headwall at the outlet opening. Those installations appear to be more promising. Additionally, it is believed that providing greater vertical relief between the outlet opening and the surface drainage would help keep more of the underdrain outlets open and free flowing.

With these factors in mind, other systems of outletting the water from the free-draining layer have been designed and constructed on subsequent free-draining pavement projects in West Virginia. These systems will be monitored in the future to determine which type provides optimum performance.

REFERENCES

1. L. K. Moulton. *Highway Subdrainage Design.* Final Report. Federal Highway Administration, Washington, D.C., Aug. 1980.
2. H. R. Cedergren, J. A. Arman, and K. H. O'Brien. *Development of Guidelines for the Design of Subsurface Drainage for Highway Pavement Structural Sections.* Final Report. Federal Highway Administration, Washington, D.C., Feb. 1973.

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Subbase Permeability and Pavement Performance

KEITH L. HIGHLANDS AND GARY L. HOFFMAN

This project demonstrated that open-graded, permeable, subbase materials can be designed that provide adequate constructibility and pavement support as well as good internal drainage at a competitive cost. Five types of subbases ranging from an impermeable cement stabilized material to a very permeable, uniformly graded, crushed aggregate were evaluated during this 7-yr research project. Testing indicated that the open-graded subbases had adequately high permeabilities, but the dense-graded subbase permeability was unsatisfactorily low. Deflection measurements, indicating the relative strength of the pavement structure in each subbase section, were made using a falling weight deflectometer. The lowest pavement deflections were found in the aggregate cement section. Deflections in the asphalt treated permeable material (PA No. 2B aggregate) and high permeability aggregate sections were approximately equal to each other and slightly higher than those measured in the aggregate cement section. The highest deflections were measured in the PA No. 2A aggregate section. The results of this evaluation infer that dowel looseness, pavement temperature, loading magnitude, and the extent of beam-like behavior exhibited by underlying subbases all influence joint efficiency measurements. Joint efficiency appears not to be controlled by one or even a few factors at a particular site but is influenced by a combination of factors in the pavement structure and environment during testing. Roughness measurements showed smooth pavements can be built on all five subbase types. After 7 yr, no significant difference in riding characteristics exists among the five subbase sections.

The Pennsylvania Department of Transportation (PennDOT) specification for crushed, densely graded subbase (PA No. 2A aggregate) was developed over a number of years as a result of extensive testing, evaluation, and field performance monitoring. The gradation specifications were chosen to provide

- The necessary strength and stability to support construction equipment, the pavement, and subsequent traffic;
- Drainage; and
- A material that could be manufactured with adequate quality control at reasonable cost (*1*).

The specification was developed as a compromise that would best meet the above criteria.

Numerous problems of premature pavement and shoulder distress were reported to be a result of excess water in the pavement system, and questions were raised as to whether PA No. 2A subbase was adequate to provide sufficient drainage for the pavement system. Laboratory permeability tests of the PA No. 2A subbase indicate a range in the coefficient of permeability of 2.8 to 0.028 ft per day (0.001 to 0.00001 cm/s). This range of permeability represents a very slow to nearly impermeable material. Field permeability of the subbase is varied because of in situ gradation variations at the same job site and appears to be higher in some cases than indicated by the laboratory tests. Field permeability tests conducted as part of a nationwide study by L. K. Moulton and Roger Seals of West Virginia University on PennDOT subbase indicate permeabilities in the range of 280 to 28 ft per day (0.1 to 0.01 cm/s) (*2*).

This experimental project was originally devised to demonstrate the feasibility of providing good construction and pavement support as well as good internal drainage at a competitive cost to PA No. 2A subbase. An additional, long-term objective of this research project was to determine the significance of the permeability of subbase materials on pavement performance. The subbases were chosen to represent a range in permeability from impermeable to very permeable.

A more detailed discussion of the construction and material testing of the subbases on this project may be found in PennDOT's Interim Report (*1*) and Final Report (*3*) on Research Project No. 79-3. Some information included in those earlier reports is again presented in this report.

Based on positive interim results obtained since this research project began, PennDOT has changed its specifications to require open-graded subbase (OGS) as an interlayer between rigid pavements and PA No. 2A aggregate subbase.

SITE DETAILS

The project field site is located on PA Traffic Route 66 (Pennsylvania Legislated Route 203) and U.S. Traffic Route 422 (Pennsylvania Legislated Route 1037) in Armstrong County, Pennsylvania. These routes have approximately 10,000 average daily traffic (ADT) with 7 percent trucks. A location map showing the project location is shown in Figure 1.

Five sections of base/subbase materials representing a range of permeability conditions from impermeable to very permeable were constructed. Following are listed the five material sections. (They are also shown by number in Figure 2.)

1. Aggregate cement,
2. Asphalt treated permeable material,
3. PA No. 2B aggregate subbase,
4. High permeability subbase, and
5. PA No. 2B aggregate.

Pennsylvania Department of Transportation, Bureau of Bridge and Roadway Technology, Room 1009, Transportation and Safety Building, Harrisburg, Pa. 17120.

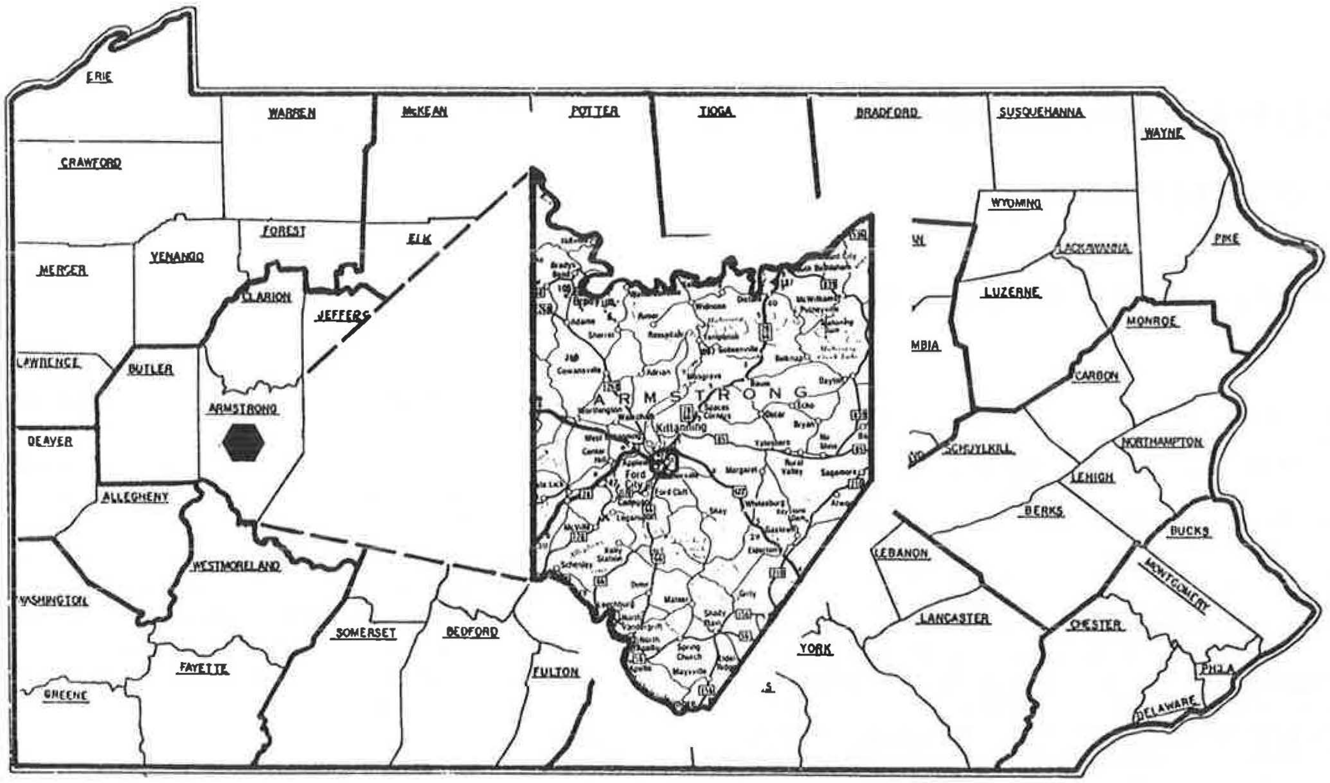

FIGURE 1 Project site location.

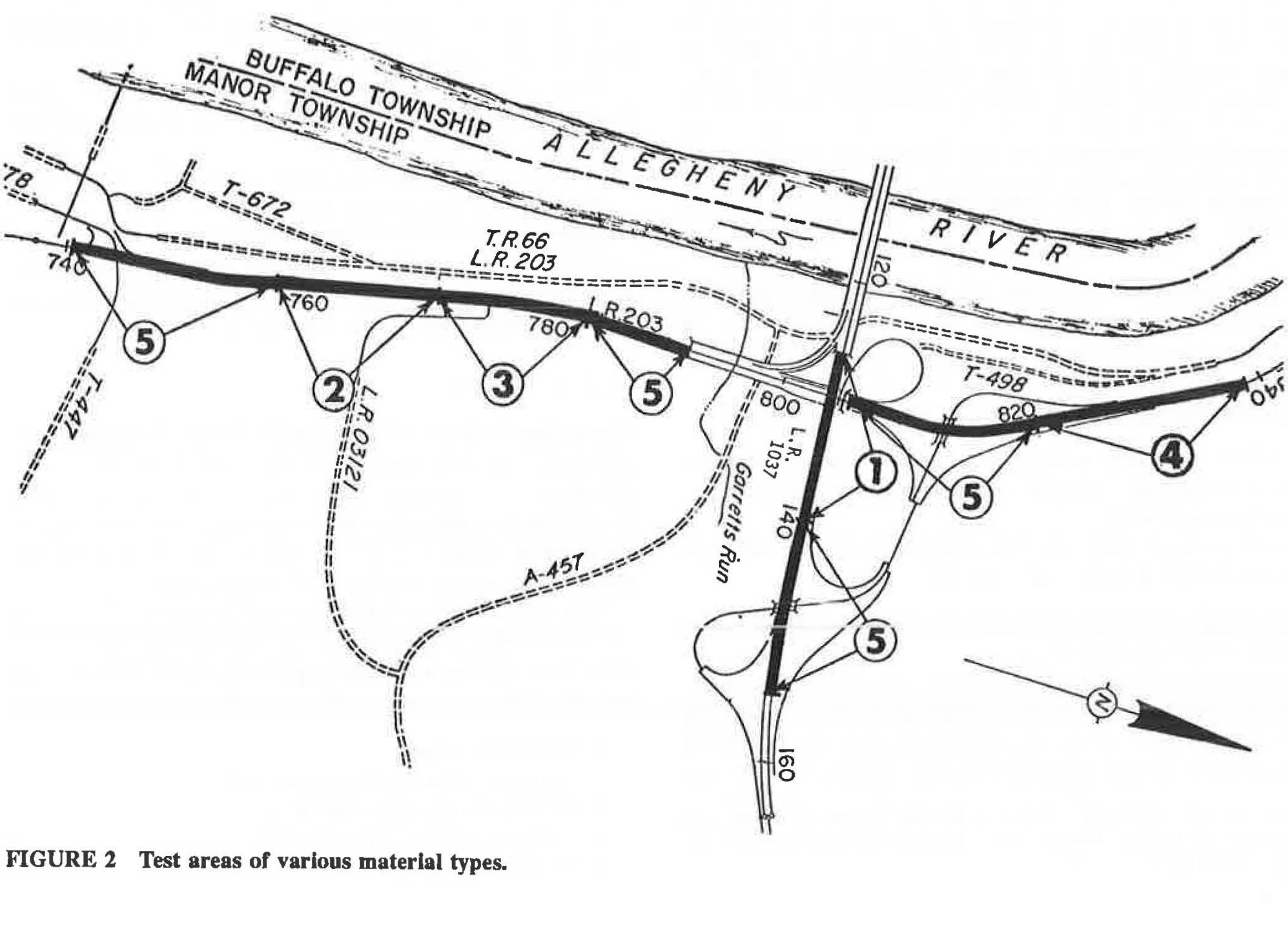

FIGURE 2 Test areas of various material types.

The standard design used for the control sections included placement of 10 in. of reinforced concrete pavement (RCP) on 13 in. of dense-graded aggregate subbase (PA No. 2A subbase). Graphite-coated round dowel bars were placed at each transverse joint. The dowels were 1¼ in. in diameter, 18 in. long, and spaced 1 ft apart. In the experimental sections, other materials were placed as an interlayer between the PA No. 2A subbase and the RCP, as shown in the pavement cross sections (Figures 3 through 7). The total thickness of the experimental interlayer and the subbase was 13 in. for all sections. Each experimental section was between 1,000 and 1,700 ft long and was constructed in adjacent sections in both the northbound and southbound lanes of the four-lane divided highway.

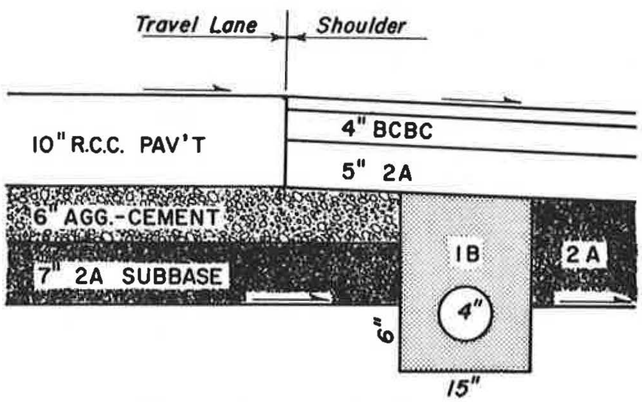

FIGURE 3 Pavement cross section (aggregate cement test area).

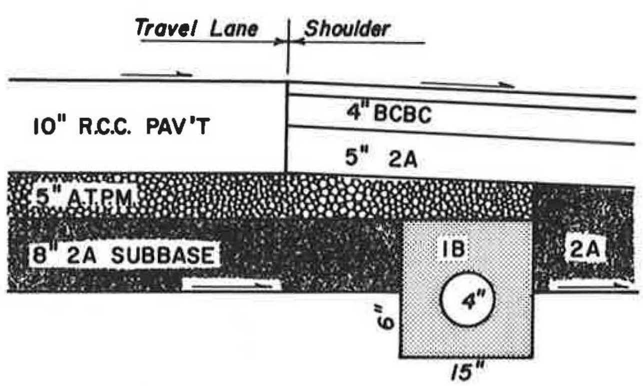

FIGURE 4 Pavement cross section (asphalt treated permeable material test area).

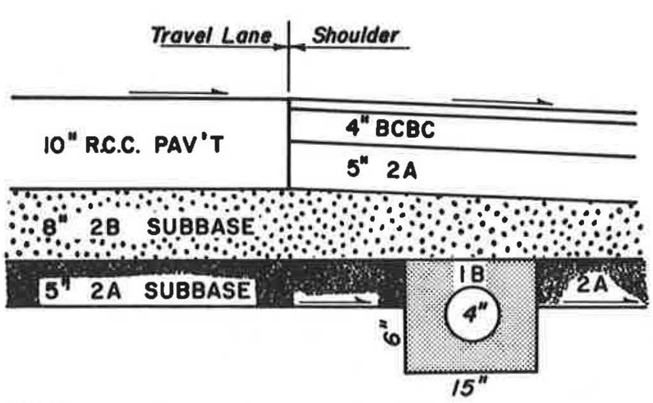

FIGURE 5 Pavement cross section (2B aggregate test area).

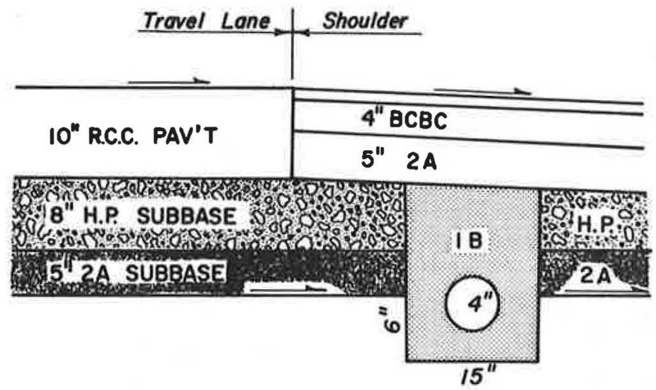

FIGURE 6 Pavement cross section (high permeability test area).

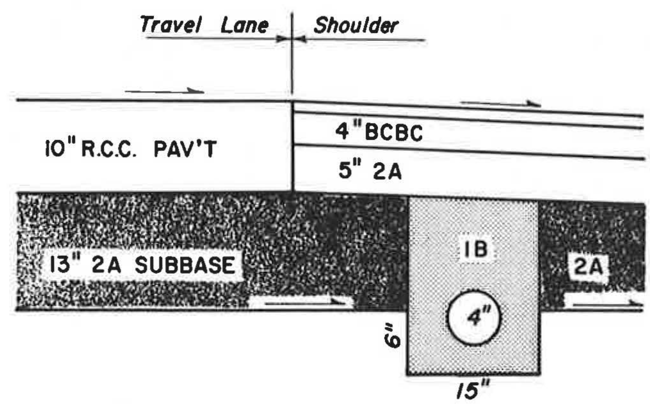

FIGURE 7 Pavement cross section (2A aggregate test area).

MATERIAL PROPERTIES

Laboratory Testing

Laboratory testing was done to determine the particle size distribution curves, the maximum dry densities, and the corresponding permeabilities of each of the five material types. California bearing ratio (CBR) testing was also performed to determine the relative difference in stability between the PA No. 2A aggregate subbase and the high permeability material. All aggregate material was a crushed glacial sand and gravel that was shipped from Davison Sand and Gravel's Tarrtown Flats source.

The specified gradation limits (dashed lines) are shown along with the actual particle size distribution curves in Figures 8 through 12, respectively, for the aggregate cement gradation, the asphalt treated permeable material (ATPM), the unstabilized PA No. 2B (2B) gradation, the high permeability (HP) gradation, and the PA No. 2A (2A) gradation. The ATPM aggregate gradation is equivalent to the 2B gradation. The 2A and the HP materials are both well-graded, but the HP material has coarser fragments than the 2A material throughout its entire particle size distribution. The 2B gradation band is narrow, and this material is uniformly sized. The 2B gradation is comparable to the AASHTO 57 gradation.

Laboratory densities, porosities, and permeabilities are tabulated in Table 1. A source specific gravity of 2.61 was used for all calculations.

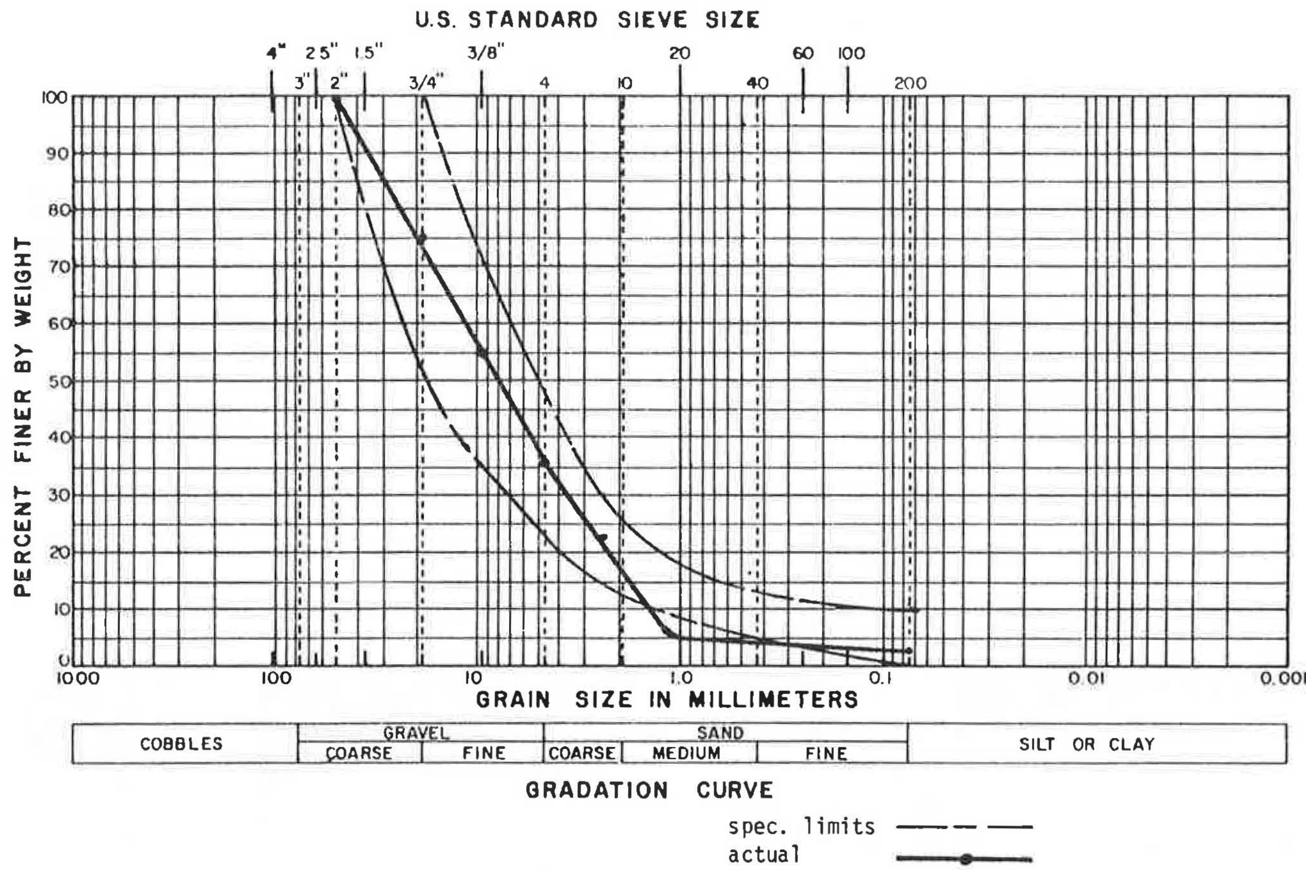

FIGURE 8 Aggregate cement (6 percent) gradation curve: aggregate only.

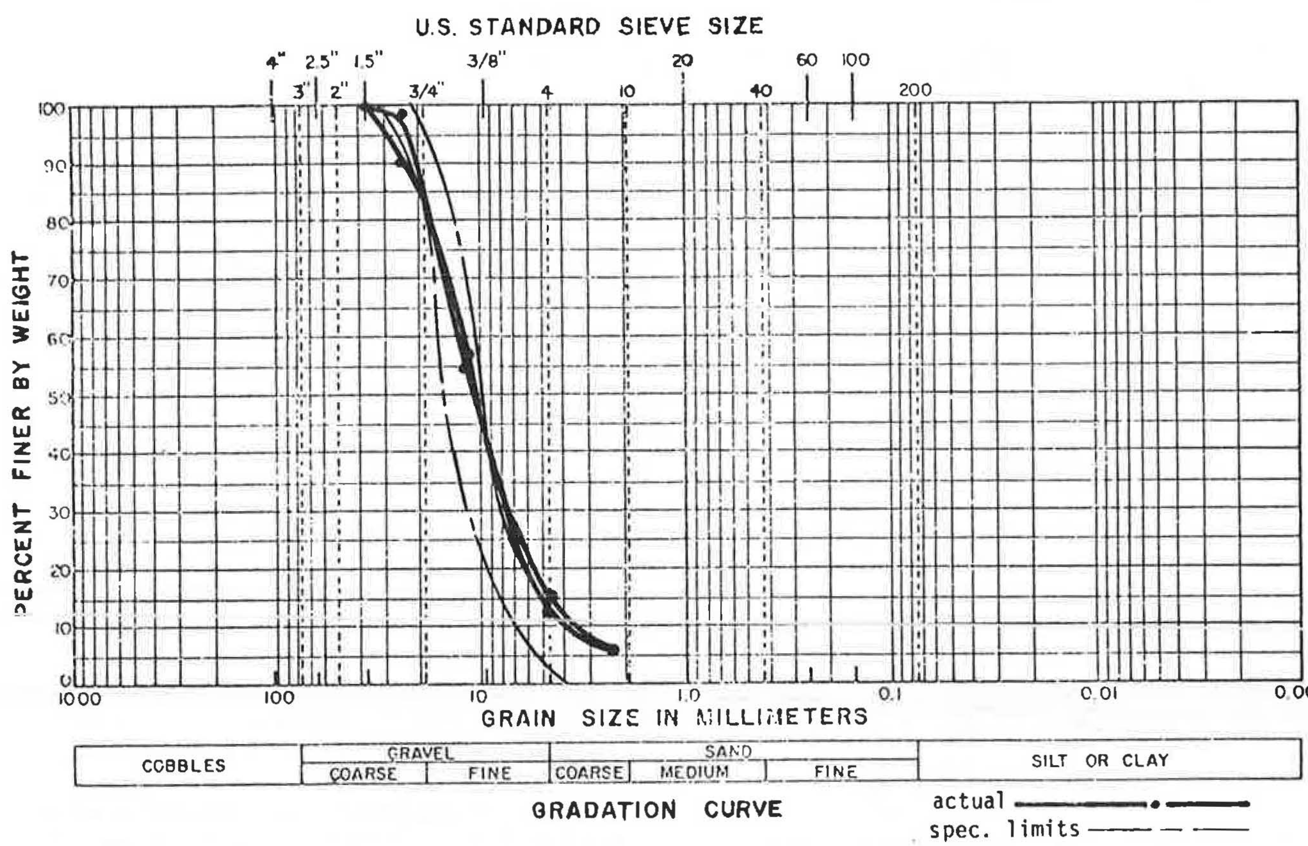

FIGURE 9 Asphalt treated permeable material gradation curve.

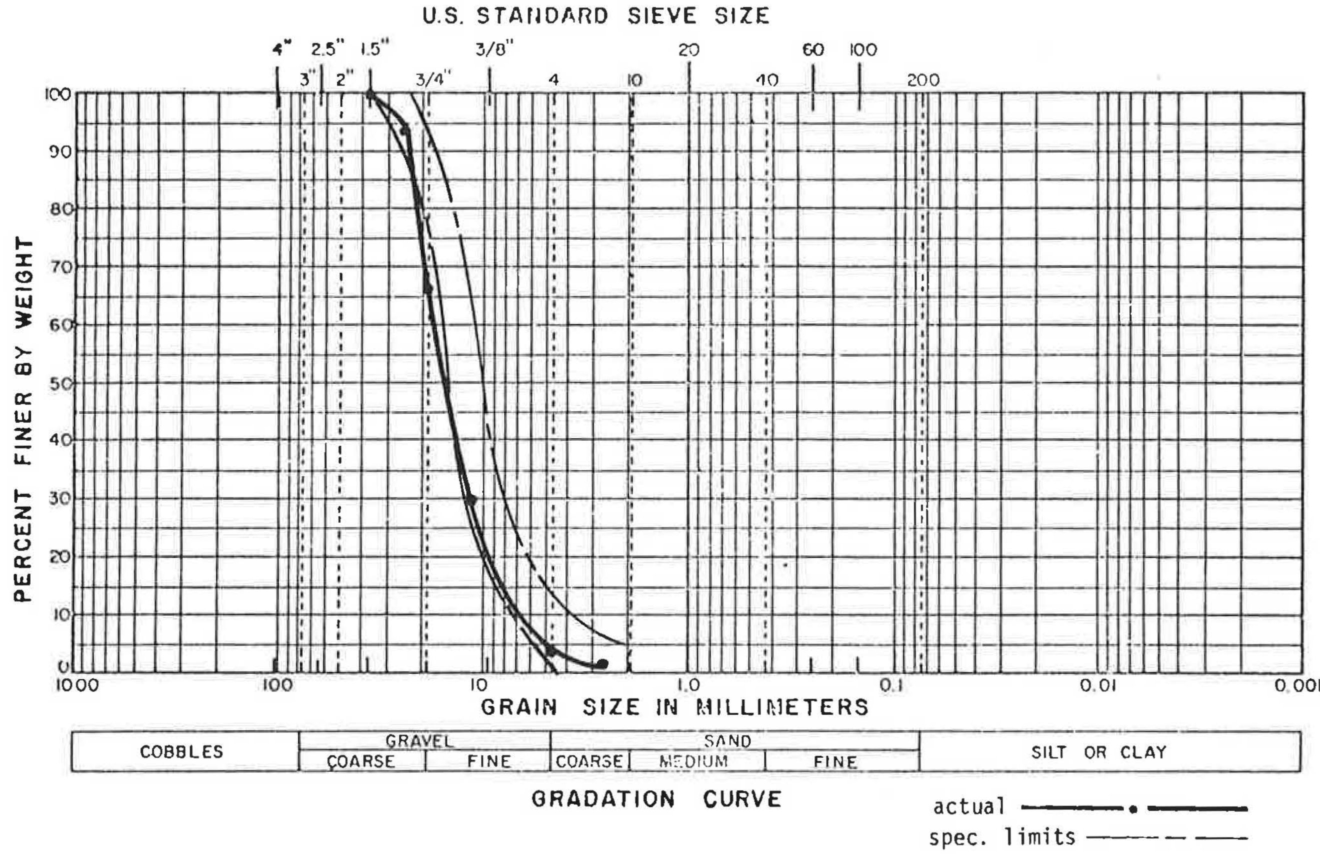

FIGURE 10 PA No. 2B gradation curve.

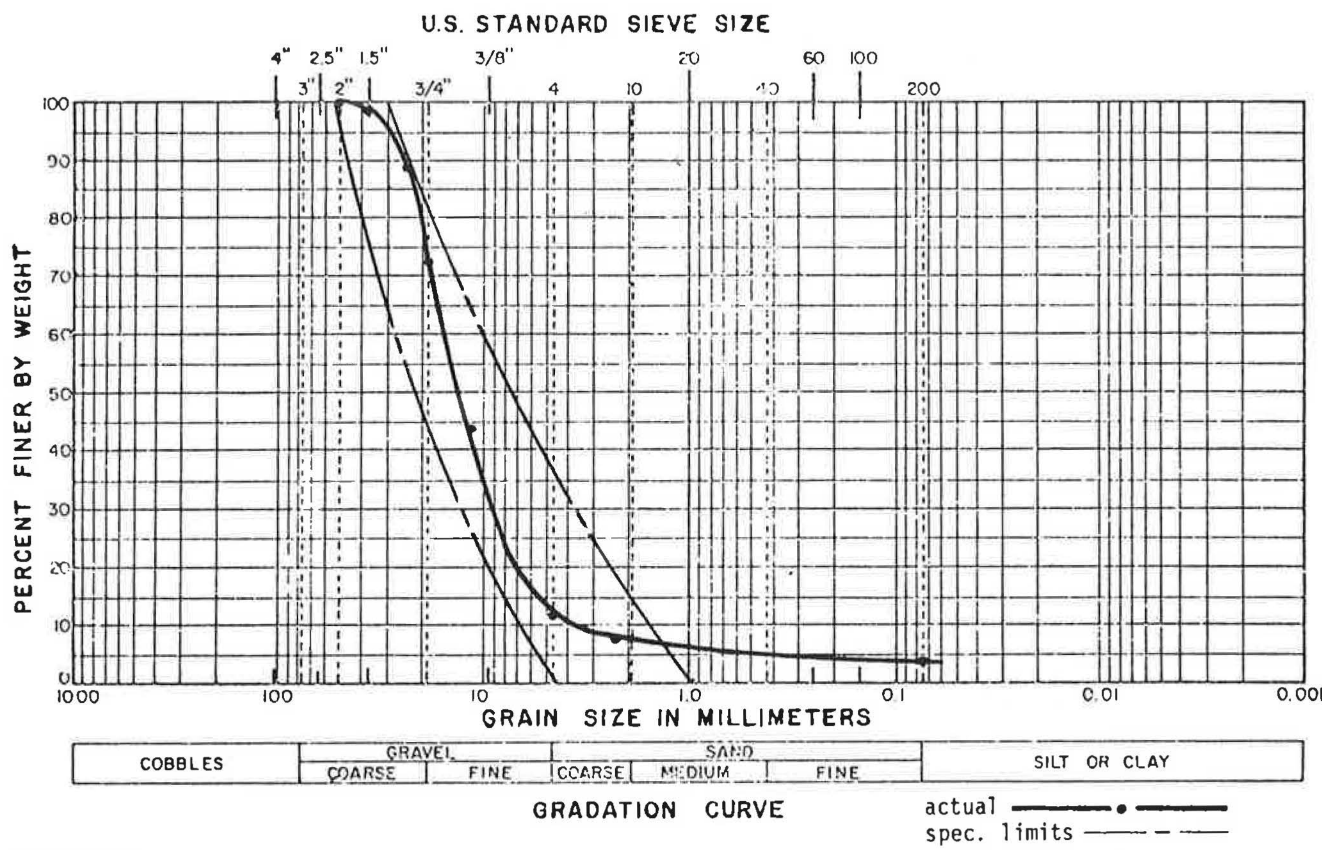

FIGURE 11 High permeability gradation curve.

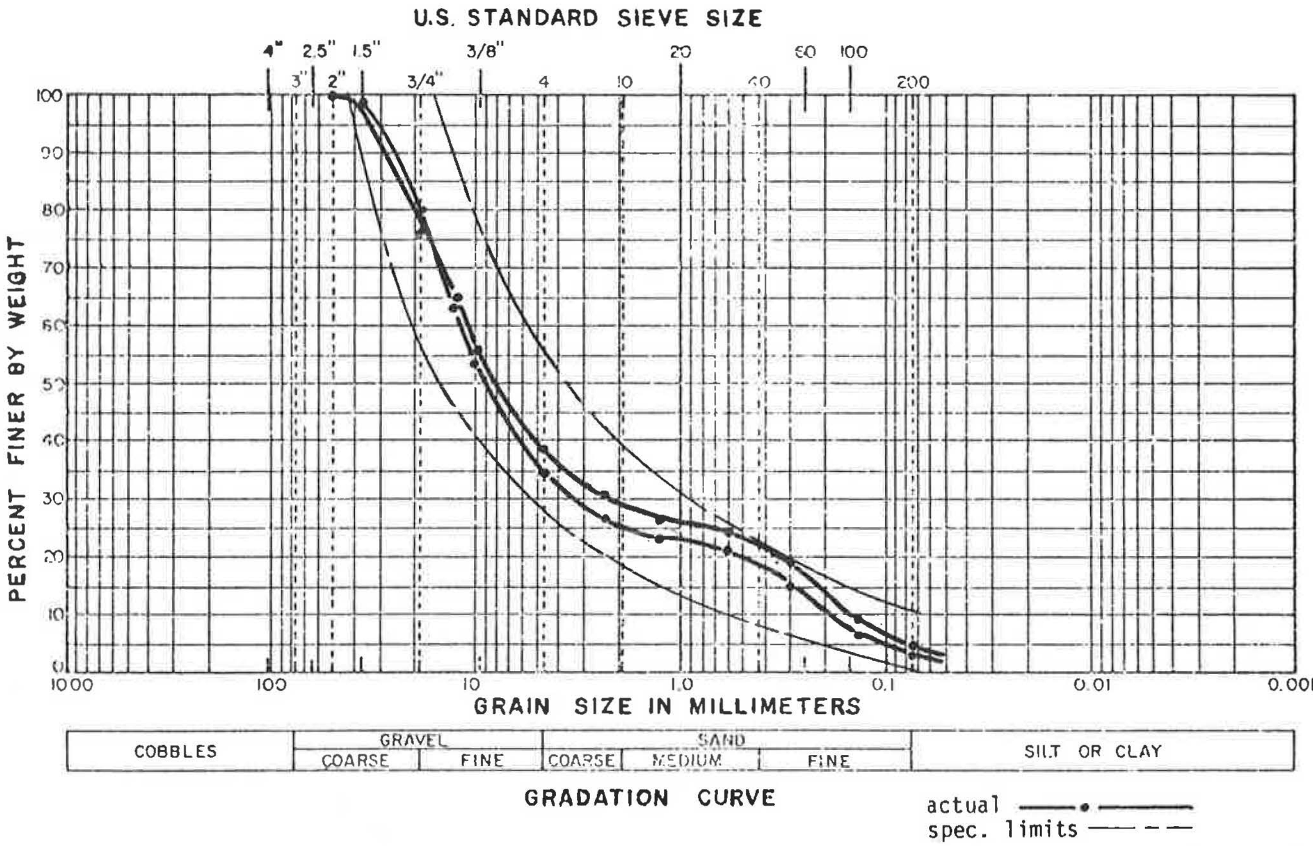

FIGURE 12 PA No. 2A gradation curve.

TABLE 1 LAB AND FIELD SUBBASE MATERIAL PROPERTIES

Subbase Type	Laboratory Permeability K ft./day	Field Permeabilities K1 ft./day	K2 ft./day	Lab. $d_{max.}$ (pcf)	Field $d_{max.}$ (pcf)	Lab. $n_{min.}$ (%)	Field $n_{min.}$ (%)	Field e
Aggregate Cement	2.83×10^{-4} (1)	(7)	(7)	138.1	138.1 (4)	16	16 (4)	0.19 (4)
ATPM	6.519×10^{3} (2)	5.39×10^{3}	6.07×10^{3}	112.7	106.9	31	33	0.51
2B Aggregate	2.15×10^{4} (2)	7.74×10^{3}	2.39×10^{4}	102.9	93.2 (5)	37	43 (5)	0.75 (5)
H.P.	1.81×10^{4} (2)	1.73×10^{4}	1.78×10^{4}	110.0	100.0	32	39	0.63
2A Aggregate	1.22 (3)	3.97×10^{1} (6)	1.79×10^{1} (6)	124.9	125.4	23	23	0.30

(1) Triaxial test permeability
(2) Fabricated falling head test
(3) Standard constant head permeameter
(4) Data obtained from mix design testing
(5) Data derived from field concrete design data
(6) Due to limitations of test equipment, field permeability measurements in 2A Aggregate may not be accurate
(7) No measurements because permeabilities were below the lower testing capabilities of the testing equipment

K = Permeability
K1 & K2 - Permeabilities in orthogonal (90 degrees apart) directions
d = Dry density
n = porosity
e = void ratio

Naturally, the stabilized aggregate cement base material had the highest maximum density, lowest porosity, and slowest permeability. The well-graded 2A subbase material (control) had the next highest maximum density, a low porosity, and a slow permeability. The ATPM and HP materials had intermediate maximum densities, porosities, and permeabilities. The unstabilized 2B material had the lowest maximum density, highest porosity, and fastest permeability. Figure 13 depicts the relationship between porosity (maximum density) and permeability.

CBR testing showed that the 2A subbase had values ranging between 80 percent and 85 percent; the HP material had values around 60 percent. Even though the relative stability of the more permeable HP material was notably less than the 2A material, it was substantial and should provide satisfactory performance when confined.

Field Testing

Field permeability testing was performed using the field permeability test device (FPTD) developed by Lyle Moulton and Roger Seals at West Virginia University. Field permeability test results are shown in Table 1. All test sections were located in the southbound lanes. In-place density measurements were made with a Troxler nuclear gauge at the FPTD test locations. Because of the high void ratio and unconfined instability of the 2B aggregate, field densities were not obtained in this material; however, a density was approximated from laboratory design

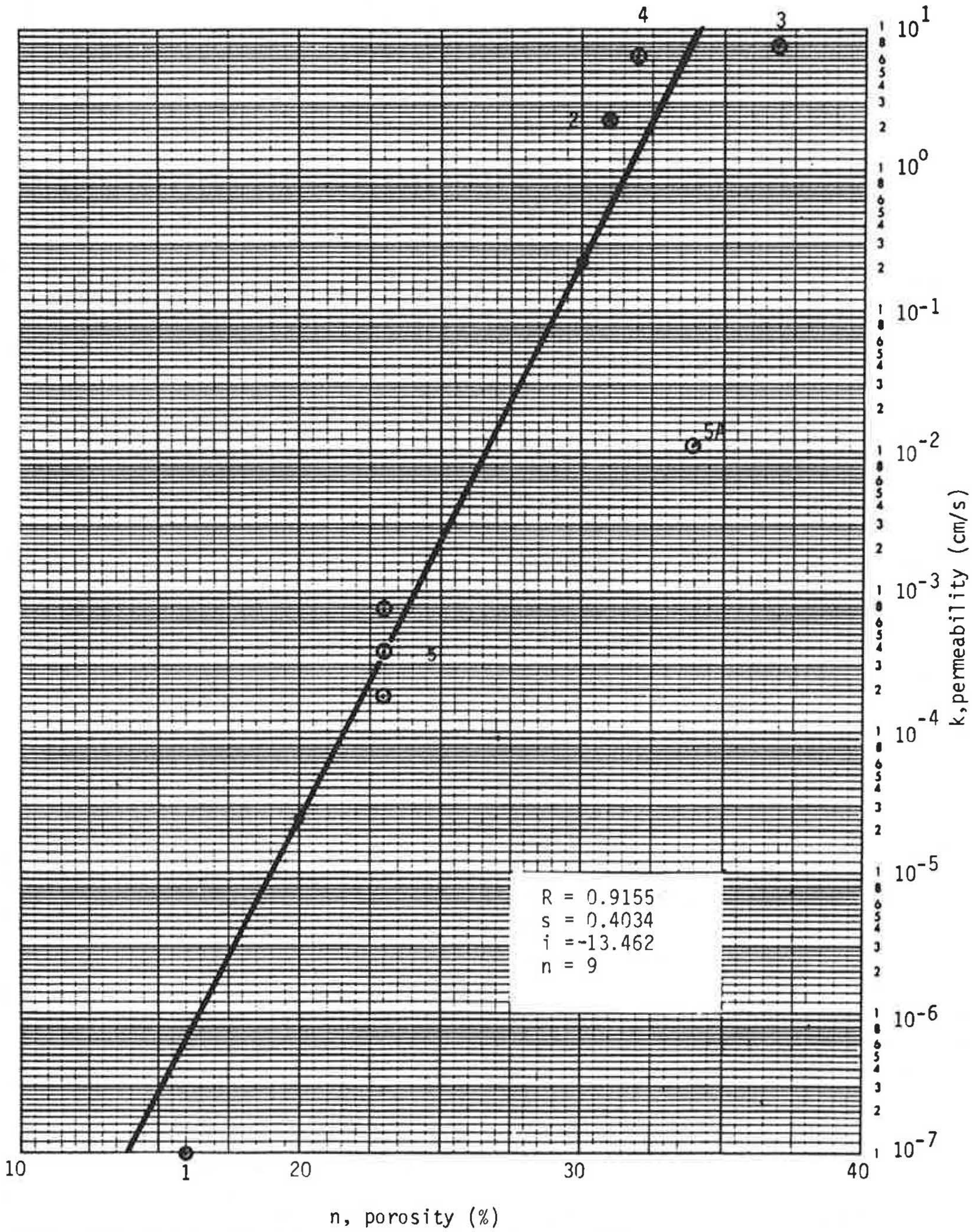

FIGURE 13 Relationship between lab porosity and permeability.

data. Void ratios (*e*) and porosities (*n*) were calculated using a source specific gravity of 2.61. These field density and porosity data are also listed in Table 1.

CONSTRUCTION DETAILS

The entire job, which included the experimental sections, was bid on a competitive basis. The unit prices received from the selected contractor for the different base/subbase materials are shown in Table 2.

TABLE 2 SUBBASE UNIT PRICES

		Interlayer Only	Interlayer and Subbase
1.	Aggregate Cement (6-inch)	$10.00/s.y.	$13.50/s.y.
2.	Asphalt Treated Permeable Material (5-inch)	$ 5.40/s.y.	$ 9.40/s.y.
3.	PA #2B Aggregate (8-inch)	$ 4.30/s.y.	$ 6.80/s.y.
4.	High Permeability (8-inch)	$ 4.30/s.y.	$ 6.80/s.y.
5.	PA #2A Aggregate (8-inch)	$ 4.00/s.y.	$ 6.50/s.y.

The five sections of different base/subbase materials were constructed in July of 1980 without major difficulties or delays, even though the contractor was unfamiliar with some of these materials.

The 10-in.-thick RCP was placed in August 1980 by conventional fixed form methods without incident. A 24-ft-wide (two lanes) pavement was placed monolithically.

The pavement base drain system was typical throughout the test sections. The longitudinal trench was excavated 13 in. (the depth of the subbase) away from the travel lane/shoulder edge of the pavement in both the northbound and southbound directions on tangent sections. The trenches were located on the low side of the pavement on superelevated horizontal curve sections. A perforated plastic drain pipe was then placed, and the trench was backfilled with PA No. 1B crushed aggregate ("pea" gravel). The plastic drain was 4⁵/₈ in. in diameter, semicircular, and was outletted through the slope or into drop inlets. Outlet spacings ranged from 100 to 600 ft and were typically on the order of 300 ft. In all cases, the experimental permeable layer was brought into immediate contact with the PA No. 1B trench backfill material to ensure a continuous flow path. It is crucial for free-draining bases to always be outletted with positive drainage systems.

More detailed information regarding the construction and material testing performed on this project may be found in PennDOT's Research Project No. 79-3 Interim Report (*1*).

PAVEMENT ROUGHNESS MEASUREMENTS AND CONDITION SURVEY

Roughness measurements were made at various times during this evaluation with the Mays ride meter. These measurements represent the response of the vehicle to road roughness and indicate both the relative paving control afforded by the various bases in the initial measurements and the pavement performance under traffic loadings in the longer term measurements. The average pavement serviceability indices (PSI) for the pavements over the subbase sections are listed in Table 3.

TABLE 3 AVERAGE PAVEMENT SERVICEABILITY INDICES

	Subbase Type	Initial 1980	1981	1986	1987
1.	Aggregate Cement	4.0	4.1	3.0	3.4
2.	Asphalt Treated Permeable Material	4.1	4.2	3.3	3.6
3.	PA #2B Aggregate	3.9	4.0	3.2	3.3
4.	High Permeability	3.8	4.0	3.2	3.4
5.	PA #2A Aggregate	3.8	4.0	3.2	3.6

There were no extreme differences among the PSI values measured for each section at any time during this evaluation. The average PSI values of all the sections were slightly higher at the 1-yr testing point. The PSI values of all the sections were lower in 1986 than at the 1-yr testing point but were slightly higher again during the 1987 testing than during the 1986 testing. The slight increase in pavement rideability in the first year can be explained by the wearing of the surface texture and seating of the pavement under traffic.

The initial roughness measurements indicate that relatively smooth pavements can be constructed on all five subbase types included in this study. The comparable roughness measurements obtained in each section during each testing cycle do not indicate any significant influence of subbase type on short-term pavement riding characteristics. If any of these subbases fail to provide adequate pavement support in the future, the pavement distresses which result will certainly adversely affect the rideability of the pavement.

A walking visual survey of the entire pavement over the experimental subbase sections revealed that the pavement was generally in extremely good condition. One transverse crack was found between the 2A aggregate and HP sections. This crack was probably caused by the discontinuity in subbase support, because it occurred in the transition zone from one type of subbase to another.

The only other significant cracking was in the aggregate cement base section. Approximately 25 percent of the slabs in this section had mid-point transverse cracks. These cracks were probably shrinkage cracks. They occurred in this section and not in the other sections as a result of the different frictional characteristics of the slab and subbase in the aggregate cement section at the slab/base interface. As the slabs moved in the aggregate cement section during concrete curing, the frictional forces at the slab/base interface on the slabs over the aggregate cement resisted slab movement and caused cracks to occur in the slabs.

No noticeable faulting exists. There were some occasional minor (1- to 2-in.) spalls at joints. The joints were initially sealed, but because of sealant failure, debris accumulated in some of the slab joints. As the slabs expanded with increased temperature, the incompressible debris material in the joints prevented sufficient slab expansion to alleviate the thermal stresses in the slabs. These stresses most likely caused the joint spalling.

DEFLECTION TESTING

PennDOT's falling weight deflectometer (FWD) was used to measure the deflections of the pavement slabs over each subbase section evaluated in this report. The deflection data not only indicated the deflection characteristics of rigid pavement slabs over each subbase type, but were also used to calculate joint efficiencies and determine possible underlying void locations. The analysis programs used with PennDOT's FWD to determine some of the results presented in this report may be found in the Final Report of PennDOT Research Project No. 85-23 entitled "Pavement Evaluation Procedure Utilizing the Falling Weight Deflectometer" (*4*).

The deflection testing results obtained on this project indicated that pavement temperature levels significantly influence the magnitude of deflection of jointed, rigid pavements when loaded repeatedly with the same or similar loads.

Some differences exist between pavement temperatures measured manually with handheld thermometers and those measured with PennDOT's FWD during this project's deflection testing. Usually, the manually measured and FWD measured temperatures were close. There appeared to be no definite pattern relating the differences in the two types of temperature measurements. At this time, it is not known what caused the discrepancy between the two sets of temperature readings.

The average loaded slab deflections measured at a 25-cm load drop height in 1986 and 1987 are presented in Table 4. To provide data less affected by joint "locking," average loaded slab deflection values, which exclude measurements taken when slab temperatures measured with PennDOT's FWD were above 70°F, are shown in Table 4 in parentheses. As indicated in Table 4, there was not a considerable difference in the deflection readings measured in the same sections in 1986 and 1987.

As expected, the deflection data indicate that the aggregate cement subbase is the strongest. The deflection data also indicate that the ATPM, 2B, and HP subbases all have relatively equal strengths and the 2A subbase was markedly the weakest subbase type tested.

As mentioned earlier, some differences were found between manually measured and FWD measured pavement temperatures. Usually, the two temperature readings were close. Using Spring 1987 data, a linear regression of the deflections measured at the corner of the loaded slab at each joint tested versus pavement temperature was done for each subbase section. The resulting slopes of the regression lines are presented in Table 5. All slopes are negative, indicating that deflections in jointed rigid pavements constructed over each subbase type decrease as temperatures increase. Although the preciseness of the pavement temperatures may be slightly questionable, the difference in linear regression slopes calculated between the aggregate cement section and the ATPM, 2B, and HP sections is very large, with the linear regression slopes of the latter mentioned sections being 14 to 23 times greater than that calculated in the aggregate cement section. This indicates a marked difference in the deflection behavior of pavements over stiff subbases compared to that of less stiff subbases.

It is logical that deflections in pavements over stiff subbases will be relatively less affected by pavement joint locking than deflections in pavements over weak subbases. By comparing the slopes of the linear regression lines for the various subbase material types, it becomes apparent that the pavement deflections measured in the aggregate cement section decreased less

TABLE 4 AVERAGE LOADED SLAB DEFLECTIONS

	Subbase Type	1986 Loaded Slab Deflection (10^{-6} inches) *	**	Spring, 1987 Loaded Slab Deflection (10^{-6} inches) *	**
1.	Aggregate Cement	5421	(----)	5555	(5555)
2.	Asphalt Treated Permeable Material	6372	(6560)	6556	(11465)
3.	PA #2B Aggregate	7380	(----)	6370	(7382)
4.	High Permeability	7140	(7266)	7192	(10976)
5.	PA #2A Aggregate	19642	(18650)	9943	(28668)

*Averages not in parenthesis include measurements taken at all pavement temperatures.

**Averages in parenthesis only include measurements taken when pavement temperatures were less than or equal to 70 degrees F. Temperatures were measured by PennDOT's FWD.

TABLE 5 LINEAR REGRESSION SLOPES, PAVEMENT TEMPERATURE VERSUS LOADED SLAB DEFLECTION

	Subbase Type	Linear Regression Slope (10^{-6} inches/F**)
1.	Aggregate Cement	-16
2.	Asphalt Treated Permeable Material	-295
3.	PA #2B Aggregate	-225
4.	High Permeability	-375
5.	PA #2A Aggregate	-795

*Deflections were measured during Spring, 1987.

**Pavement temperatures were measured with PennDOT's FWD.

with temperature increase than deflections measured in the other sections. The aggregate cement base already exhibited the lowest average total deflection. Conversely, the dense-graded 2A section had the highest average total deflections and the greatest regression line slope or rate of change in deflection as temperatures increased.

VOID DETECTION TESTING

Included in the software used to analyze deflection data with PennDOT's FWD is a program that determines if it is probable that a void exists under the pavement at the tested joint. The void detection analysis requires repeated deflections to be measured at a joint using three different loads. The loads used on this project were approximately 6,000 lb, 9,000 lb, and 12,000 lb.

A summary of the void detection analysis results obtained on this project during Spring 1987 deflection testing is presented in Table 6. To minimize the use of data skewed by locked pavement joints, only deflections measured when pavement temperatures were equal to or less than 70°F, as measured with PennDOT's FWD, are included in the void detection results shown in Table 6.

TABLE 6 VOID DETECTION SUMMARY

S	Subbase Type	Joint No.	Testing Year	Probability that Void Exists? Approach Slab	Leave Slab
1	Aggregate Cement	5	1987	No	No
1	Aggregate Cement	13	1987	No	No
2	Asphalt Treated Permeable Material	9	1986	No	No
2	Asphalt Treated Permeable Material	9	1987	Yes	Yes
2	Asphalt Treated Permeable Material	16	1986	No	No
3	PA #2B Aggregate	2	1987	No	No
4	High Permeability	15	1986	Yes	No
4	High Permeability	15	1987	No	No
4	High Permeability	18	1986	No	No
5A	PA #2A Aggregate	4	1986	Yes	Yes
5A	PA #2A Aggregate	16	1986	Yes	Yes
5A	PA #2A Aggregate	16	1987	Yes	Yes
5A	PA #2A Aggregate	18	1986	Yes	Yes
5A	PA #2A Aggregate	18	1987	Yes	Yes

Table 6 indicates that all joints in the 2A aggregate sections that were tested for voids probably had voids under both the approach and leave slabs. None of the other subbase sections exhibited a tendency to have voids at joints. During the 1987 testing, a probable void was found under a joint in the ATPM section. In 1986, a probable void was located under the approach slab of one joint in the HP section, but this determination is questionable. The data indicating whether a void exists were near the borderline of the criteria for determining the probability of a void existing. In 1987, no void was detected at the same joint and slab. The data were slightly on the other side of the criteria for determining the probability of void existence under a slab.

The gradation and poor drainage characteristics of 2A subbase as compared to ATPM, 2B subbase, and HP subbase make it more susceptible to void formation due to the buildup of pore pressures and the pumping of fine material. The aggregate cement base is probably too impermeable and too rigid to readily allow void formation.

JOINT EFFICIENCY

The joint efficiency results used in this evaluation were calculated using FWD deflection data. Listed in Table 7 are the average joint efficiencies calculated for FWD tested joints in each subbase section during the Spring of 1987. FWD measurements used to calculate the average joint efficiencies were not purged because of high pavement temperatures. For uniformity in sampling, only measurements made at an FWD load drop height of 25 cm were included in the Table 7 data. Thus, the impact force on each joint whose efficiency is included in Table 7 was approximately the same.

TABLE 7 AVERAGE JOINT EFFICIENCIES (25-cm LOAD DROP HEIGHT), SPRING 1987 TESTING

	Subbase Type	1986 Average Joint Efficiency (%)	1987 Average Joint Efficiency (%)
1.	Aggregate Cement	73	70
2.	Asphalt Treated Permeable Material	66	76
3.	PA #2B Aggregate	62	49
4.	High Permeability	37	54
5.	PA #2A Aggregate	94	83

As indicated in Table 7, the average joint efficiencies calculated in the more open-graded HP and 2B aggregate sections are less than the other sections, but the joints in the impermeable, stable, aggregate cement section had lower efficiencies than those in the 2A aggregate sections. The variation in average joint efficiencies presented in Table 7 implies that joint efficiency, as calculated by PennDOT's FWD, may vary with subbase type. With the information presented in Table 7, the correlating factors between joint efficiency and subbase type cannot be definitely determined.

Certainly, a major factor determining the efficiency of pavement joints is the effectiveness of the load transfer devices constructed at the joints. The round dowel bars placed at each transverse joint were $1^1/_4$ in. in diameter, 18 in. long, and spaced 1 ft apart. Since identical load transfer devices and construction techniques were used for all the joints in all sections of this project, it is reasonable to expect little or no difference in the average joint efficiencies of the various sections as a result of the load transfer devices.

The formula currently being used to calculate joint efficiencies with PennDOT's FWD is (*4*)

$$\textit{Joint Efficiency} = \frac{\textit{Deflection of Nonloaded Slab} \times 100}{\textit{Deflection of Loaded Slab}}$$

In 1958, Teller and Cashell reported that dowel looseness, caused by coatings used to prevent bond, concrete air or water voids, concrete shrinkage during hardening, and wear of the concrete surrounding the dowel by repetitive loading, is a significant variable affecting joint efficiency (*5*, p. 26). They also stated that they consider dowel looseness to include all conditions preventing dowels from offering full load resistance (*5*, p. 16). In the same report (*5*, p. 26), Teller and Cashell stated,

> It is obvious that a dowel or dowel system does not begin to function at maximum efficiency until all looseness is taken up by the deflecting pavement on the loaded side of the joint farther than would be necessary if initial looseness were not present.

All dowels in all sections constructed on this project had a graphite bond breaker applied to the dowels in the field. The relatively small traffic loadings should not have caused excessive concrete wear. Probably, most of the existing dowel looseness was built into the pavement and should be approximately the same for all sections.

Teller and Cashell stated further (*5*, p. 17),

> It is apparent that the load-transfer system is much more effective when the slab-end deflection is relatively large. . . . It is of interest that once the play and looseness of the system is taken up the effectiveness is relatively high. . . .

During the Spring 1987 FWD testing, pavements over the 2A subbase had the highest deflections. The dowels in the 2A sections were deflected beyond the amount necessary to "take up" dowel looseness farther and for a greater percentage of their total deflections than dowels in the other subbase sections. This allowed the load transfer devices in the 2A aggregate sections to work at maximum efficiency during a larger portion of the deflections of the slabs, resulting in a higher overall average percent joint efficiency for the joints in the 2A aggregate sections.

The ATPM section, 2B section, and HP section all had similar average loaded slab deflection magnitudes. They all would have deflected past the amount necessary to take up dowel looseness approximately the same amount, so the factor influencing the joints in the ATPM section to have a higher joint efficiency than those in the 2B aggregate and HP sections was probably not dowel looseness. Because other factors affect pavement performance and were apparently influencing joint efficiency calculations in the other subbase sections, it was realized that dowel looseness was probably not the only factor causing higher joint efficiency results in the 2A aggregate section.

It was decided that additional FWD testing should be done to further define the factors influencing joint efficiency measurements. This additional FWD testing took place during the Fall of 1987. During this testing, the loads and drop heights of the FWD were adjusted so that approximately equal deflections were obtained in each of the subbase sections. This should have reduced the effect of dowel looseness on joint efficiency results since pavement slabs in each subbase section would be deflected past the amount required to take up dowel looseness the same amount. The results of the Fall 1987 joint efficiency testing are shown in Table 8.

The two types of subbases on this project which were bound with cementitious materials are aggregate cement and ATPM. The 2A aggregate subbase is not a bound material but compacts very tightly because of the amount of fine material included in its gradation. The HP and 2B aggregate subbases are not bound and do not have the amount of fine material in their gradations that the 2A subbase does. Thus, they do not compact as tightly as the 2A aggregate subbase does.

During the equal deflection testing, joint efficiency results in the unbound 2A aggregate section were similar to those obtained in the two bound subbase sections. This inferred dowel looseness probably did play a part in causing higher joint efficiency results in the 2A aggregate section during the Spring 1987 testing.

Even with the equal deflections imparted on the pavement, there remained a significant difference between the joint efficiencies measured in the relatively open-graded 2B and HP subbase sections and the 2A aggregate and bound subbase sections. This may be explained by considering the bound subbases and the tightly compacted 2A aggregate subbase to deflect in more of a beam-like manner than the 2B and HP subbases. The more beam-like subbase deflections would allow the subbase under the immediately adjacent nonloaded slab to deflect when a load is imparted on a slab. The higher deflections in the 2A aggregate and bound subbases would cause a lack of pavement support, resulting in higher deflections in the nonloaded slabs over the 2A aggregate and the bound subbases. Higher deflections in nonloaded slabs result in higher joint efficiencies. Hence, if all other factors influencing joint efficiency are equal, lower joint efficiencies should be found in pavements over bound or tightly compacted subbases than over unbound, relatively loosely compacted, more open-graded subbases.

Pavement temperature is also believed to affect the magnitude of deflections in adjacent nonloaded slabs and, hence, joint efficiency calculations. Two sets of measurements, approximately equal in deflection magnitude, were made on 2 different days in the ATPM section. There was approximately 12°F difference in the average pavement temperatures measured during testing in the ATPM section on these 2 days. The average joint efficiency calculated during the higher temperature deflection testing was approximately 11 percent higher than that calculated during the lower temperature deflection

TABLE 8 AVERAGE JOINT EFFICIENCIES, APPROXIMATELY EQUAL DEFLECTIONS IN EACH SECTION, FALL 1987 TESTING

	Subbase Type	Average Pavement Temperature During Testing (Degrees F*)	Average Loaded Slab Deflection (10^{-6} inches)	Average Joint Efficiency (%)
1.	Aggregate Cement	68	12,230	70
** 2a.	Asphalt Treated Permeable Material	57	13,620	81
** 2b.	Asphalt Treated Permeable Material	45	12,760	70
3.	PA #2B Aggregate	70	12,390	47
4.	High Permeability	65	13,630	52
5.	PA #2A Aggregate	68	12,680	75

*Pavement temperatures were measured with PennDOT's FWD.

**For comparison purposes, two sets of equal deflection measurements were obtained on two different days in the Asphalt Treated Permeable Material section.

testing. This indicates the significant effect pavement temperatures have on deflections in adjacent nonloaded slabs and the joint efficiencies that are calculated using those deflections. Increased joint locking at higher pavement temperatures causes relatively larger deflections in adjacent nonloaded slabs which results in higher joint efficiencies.

Another factor which could affect joint efficiency results is the magnitude of the load applied during the FWD deflection testing. If higher loads are applied during testing, the pavement will be deflected farther past the amount necessary to take up dowel looseness, allowing load transfer devices to work at maximum efficiency during a larger percentage of the total slab deflection. As discussed earlier in this report, higher joint efficiencies will result. The effect different pavement loadings have on joint efficiency should be kept in mind when interpreting FWD joint efficiency data.

As discussed in this report, joint efficiency appears to be affected by dowel looseness, pavement temperature, loading magnitude, and the extent of beam-like behavior exhibited by underlying subbase. It is not presumed that these are the only factors that influence joint efficiency. Joint efficiency appears not to be controlled by one or even a few factors at a particular site but to be influenced by a combination of factors in the pavement structure and environment during testing.

It should be kept in mind that the FWD joint efficiency calculation method of dividing the nonloaded slab deflection by the loaded slab deflection does not necessarily indicate only the load transfer occurring across the slabs or even how well the pavement's load transfer devices are performing. As discussed in this report, if some subbases do not deflect in a beam-like manner as much as others, they will support nonloaded slabs better. Lower joint efficiencies will be determined at joints in these better supported pavements. Lower joint efficiencies influenced by the beam-like deflection characteristics of subbases do not indicate a pavement support problem or that load transfer devices are working less effectively.

It is realized that PennDOT's FWD joint efficiency formula is not the only available formula for calculating load transfer efficiency. A load transfer measuring formula that takes into account the variety of factors influencing joint efficiency could make joint efficiencies calculated by a FWD more useful to engineers determining required joint rehabilitation.

PENNDOT'S OPEN-GRADED SUBBASE

PennDOT now uses open-graded subbase (OGS) as an interlayer between rigid pavements and 2A aggregate. OGS was developed to provide an aggregate layer under pavements that is more free draining than 2A aggregate, does not "pump" fines, and is capable of supporting pavement loads. To optimize manufacturing, construction stability, and permeability of the OGS material, it was decided to make the OGS permeability approximately 500 ft per day (0.18 cm/s). Field measurements indicate a 500 ft/day to 1,500 ft per day (0.18 to 0.53 cm/s) permeability in OGS.

The gradation curve for OGS is shown in Figure 14. The OGS gradation is in between the gradations of the high permeability and 2B aggregate subbases mentioned in this report.

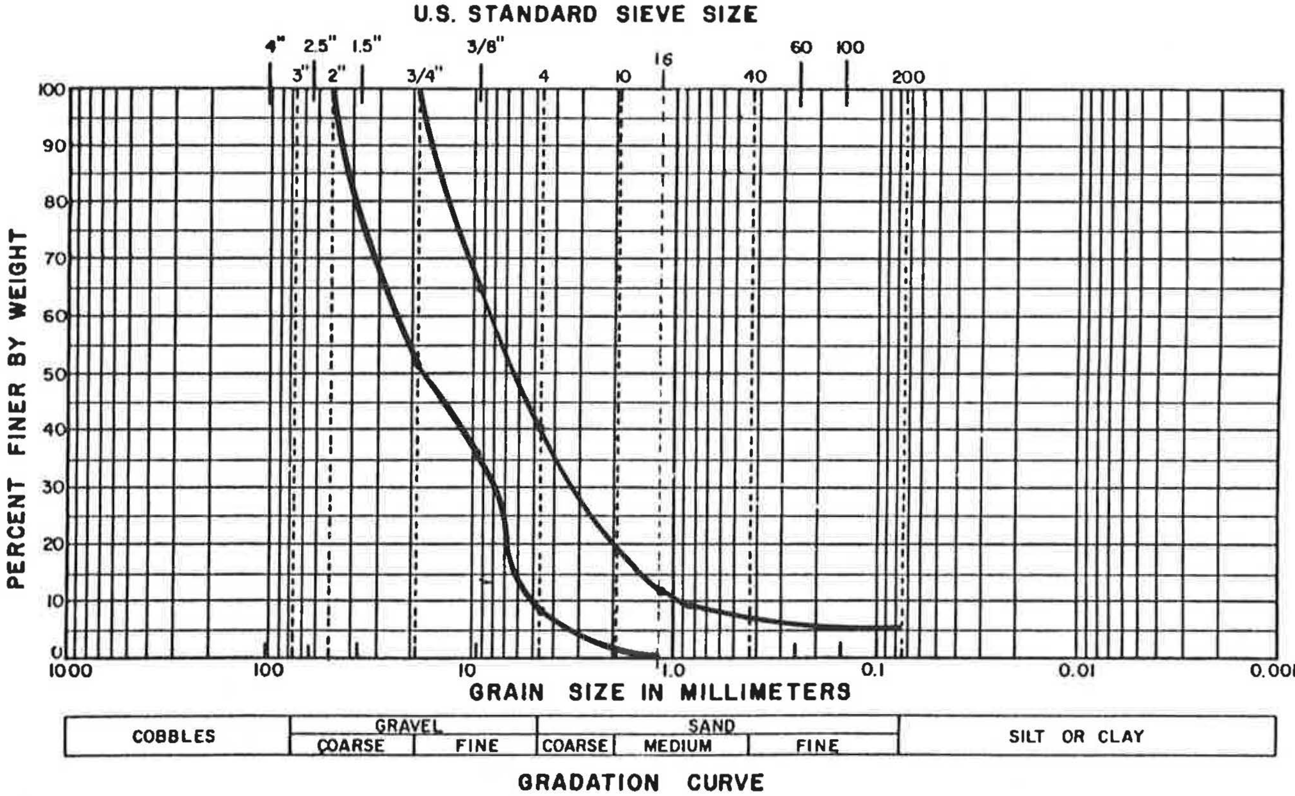

FIGURE 14 PA No. OGS gradation curve.

As a result of the gradation, most aggregate suppliers can produce OGS with one straight crusher run and do not have to blend different sizes of aggregates. This provides fewer OGS production steps which allows lower costs.

OGS has a greater tendency to segregate during handling and placing operations than does denser-graded subbase. Because of this, some contractors are having trouble meeting gradation requirements for OGS when random sampling is done on material placed on grade. PennDOT is considering looking at another point of sampling OGS to determine if it meets gradation requirements. This other point of testing will be earlier in the placing operation where the effects of segregation due to handling have not been introduced. It is believed that the fine end of the gradation band is not critical from a confined stability standpoint and that this portion of the matrix will drop to the bottom inch of the OGS layer, probably during the construction sequence of first hydraulic loading, anyway. However, these fines will probably improve the compactibility and construction stability of the material.

Contractors have demonstrated on a few major jobs their ability to achieve good compaction and stability of the OGS and successfully pave on it using both slip-form and fixed-form techniques.

CONCLUSIONS

- Subbase material with significantly high permeabilities (three or more orders of magnitude) can be produced with adequate quality control at a competitive cost. The contract price for the original standard design of 13 in. of 2A subbase was \$6.50/sq yd. The substitution of 8 in. of 2B or HP material as an interlayer on top of 5 in. of subbase increased the comparable cost to \$6.80/sq yd—about 5 percent. The substitution of 5 in. of ATPM on top of 8 in. of subbase increased the comparable cost to \$9.40/sq yd—about 45 percent over the cost of 13 in. of subbase. This ATPM cost increase would not be as great in a flexible pavement design where a higher structural coefficient and lesser required thickness, as compared to the standard unbound aggregate subbase, would be used.
- Adequate stability to support construction equipment was provided by the more porous, open-graded base materials. All sections of various bases were constructed without major difficulties or delays even though the contractor was unfamiliar with some of these materials. Pavement roughness measurements on the new reinforced concrete pavement indicated that the stabilized aggregate cement and ATPM sections had PSI values 0.2 to 0.3 higher than the unstabilized or unbounded sections. The PSI values of the pavement in the unstabilized open-graded materials sections were approximately equal to the sections with the previous 2A dense graded subbase. These roughness comparisons were similar after 15 mo, 6 yr, and 7 yr of service life with only 0.2 to 0.3 variation in PSI among the sections during each respective testing.
- The three open-graded materials had adequately high permeabilities, but the permeability of the 2A subbase was unsatisfactorily low. The more porous, open-graded ATPM, 2B, and HP materials exhibited field permeabilities on the order of 2.8×10^3 ft per day (10^0 cm/s), while the standard 2A subbase had measured permeabilities of 28 ft per day to 0.28 ft per day (10^{-2} cm/s to 10^{-4} cm/s). Excellent relationships existed between measured laboratory and field permeabilities for the

same materials. Field testing results indicated that permeabilities measured in two orthogonal directions at the same location generally were not significantly different. Permeabilities varied by as much as one order of magnitude within a material section because of segregation resulting from placement practices. The more fines that exist in the material or the more "gap-graded" the material is, the greater the propensity for segregation to occur.

- Visual surveys of pavement surface conditions indicate that all sections are in extremely good condition, albeit the truck traffic frequencies are relatively small. Only minor joint spalling and no faulting were noted.
- The results of this evaluation infer that dowel looseness, pavement temperature, loading magnitude, and the extent of beam-like behavior exhibited by underlying subbases all influence joint efficiency measurements. Joint efficiency appears not to be controlled by one or even a few factors at a particular site but is affected by and influenced by a combination of factors in the pavement structure and environment during testing.
- Average total deflection measurements, indicating relative strengths of the pavement sections, show the aggregate cement section to have the lowest deflections, while the deflections in the ATPM, 2B aggregate, and HP sections were approximately equal to each other but slightly higher than the deflections measured in the aggregate cement section. The 2A section had markedly the highest total deflections. These data indicate that the open-graded subbases should out-perform the dense-graded 2A material from a structural standpoint under the same loading conditions.
- An assessment of the probability of voids existing under pavement joints made from FWD deflection results indicates that voids probably already exist under all the joints tested in the 2A dense-graded aggregate sections. There was no strong tendency shown in the data obtained during this study to indicate voids frequently exist in the other subbase material sections. These data support the achievement of one of PennDOT's main objectives by switching to open-graded materials, that is, reducing the pumping of fines, which creates voids and ultimately causes loss of support, faulting, and deterioration of the slabs at the joints.
- PennDOT has changed its specifications and standards to require the use of open-graded subbase (OGS) interlayers immediately beneath rigid pavements. This change was based on the early results of this project. The intermediate range (7 yr) results relating the performance of dense-graded and open-graded subbases continue to support PennDOT's decision to use OGS.

ACKNOWLEDGMENTS

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REFERENCES

1. Gary L. Hoffman. *Subbase Permeability and Pavement Performance.* Interim Report, Research Project No. 79-3. Pennsylvania Department of Transportation, Harrisburg, Jan. 1982.
2. L. K. Moulton and R. K. Seals. *Determination of the In-Situ Permeability of Base and Subbase Courses.* Report No. FHWA-RD-79-88. Federal Highway Administration, Washington, D.C., May 1979.
3. Keith L. Highlands and Gary L. Hoffman. *Subbase Permeability and Pavement Performance.* Final Report, Research Project No. 79-3. Pennsylvania Department of Transportation, Harrisburg, Sept. 1987.
4. W. P. Kilareski and T. E. Ozbeki. *Pavement Evaluation Procedure Utilizing the Falling Weight Deflectometer.* Final Report, Research Project No. 85-23. Pennsylvania Department of Transportation, Harrisburg, May 1986.
5. Leslie W. Teller and Harry D. Cashell. *Performance of Doweled Joints Under Repetitive Loading.* Bulletin 217. Highway Research Board, National Research Council, Washington, D.C., 1959.

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Core Flow-Capacity Requirements of Geocomposite Fin-Drain Materials Used in Pavement Subdrainage

BARRY J. DEMPSEY

A study was conducted to determine the core flow-capacity requirements of geocomposite fin-drain materials used in pavement subdrainage. The study consists of a laboratory testing program, field subdrainage outflow studies, and analysis of all data to define the fin-drain core flow-capacity requirements. Six different fin-drain materials were tested in a 24-ft-long laboratory channel to establish their core flow properties. The tests were conducted at three different entrance heads (6.3 in., 12.3 in., and 18.4 in.) and at slopes of 0, 1, 2, 3, and 4 percent. Field subdrainage outflow data were obtained from two test sites in Illinois. These data are presented in terms of volume of outflow as a function of time. The data were collected by use of tipping-bucket flow meters. A comparison was made between the observed core flow capacity of the fin-drain materials in the laboratory and the flow volume observed from field subdrainage systems. Based on this comparison, it was indicated that geocomposite fin-drain systems will be required to provide flow zone capacities in excess of 150 gal/hr at 0 percent pavement gradient and in excess of 200 gal/hr at gradients of 1 percent or greater to compare with a standard pipe and sand envelope system. When compared with a more permeable aggregate envelope system or a high-performance fin-drain system, flow zone capacities in excess of 200 gal/hr to 300 gal/hr may be desirable depending upon the pavement gradients and the number and size of joint and crack openings. Based on faulting measurements, it is indicated that a fin-drain subdrainage system can improve pavement performance to a level equivalent to or better than that for some standard systems.

One of the major changes in the new *AASHTO Guide for Design of Pavement Structures* was the provision for guidance in the design of subsurface drainage systems and for modifying the design equations to take advantage of improvements in pavement performance resulting from good drainage practices (*1*). Although it is left up to the design engineer to identify what level or quality of drainage is achieved under specific drainage conditions, a set of general definitions corresponding to different drainage levels for a pavement structure is presented in Section II of the AASHTO Guide (*1*). These drainage levels are shown in Table 1.

While some states are just beginning to initiate drainage design standards, numerous others have had design standards for a considerable period of time and have constructed many miles of pavement subdrainage. Follow-up studies in Illinois, California, and several other states have indicated that effective structural pavement drainage decreases pavement distress and increases pavement performance life (*2–4*). Generally these pavement subdrainage systems have consisted of 4-in. to 6-in. perforated pipe placed in a trench along the edge of the pavement system which is backfilled with sand or coarse aggregate envelope material. This system may or may not include the use of a geotextile. Beginning in 1983, the construction of structural pavement subdrainage using geocomposite fin-drain materials came into widespread use. Since that time many states have adopted geocomposite fin drains as an alternate to the standard circular pipe and sand or coarse aggregate envelope systems. In this paper a geocomposite fin drain is defined as a rectangular polymeric core material that is wrapped with a geotextile and that has considerable in-plane water flow capacity.

With the adoption of geocomposite fin-drain materials for pavement subsurface drainage, the question now surfacing in the construction standards is related to the level of performance needed by these materials. In the past the benchmark for drainage has been the standard circular pipe and granular envelope system. Any new drainage concept was required to provide drainage capacity and field performance equivalent to or better than the existing standard. Furthermore, it was indicated that the new concept would have to be cost competitive.

Based on the past 4 yr of pavement subdrainage construction and evaluation it appears that a geocomposite fin-drainage system can be constructed that will provide drainage capacity and field performance that are equivalent to or exceed those of the standard circular pipe at a comparative cost. With the success of the geocomposite fin system for pavement subdrainage, department of transportation design offices are now being approached with a broad range of geocomposite fin

TABLE 1 QUALITY OF DRAINAGE FOR PAVEMENT STRUCTURES (*1*)

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	Will not drain

Department of Civil Engineering, University of Illinois, Urbana, Ill. 61801.

products that are purported to meet the flow capacity, strength, and durability properties necessary for pavement edge drainage. The major problem is that there are substantial differences in the flow capacities, strengths, and performance levels of many of these products.

The general objective of this paper is to determine the core flow capacity requirements for pavement subdrainage using geocomposite fin-drain materials. The specific objectives are as follows:

- Evaluate the core flow capacities of long sections of selected geocomposite fin-drain materials in the laboratory based on channel slope and entrance head;
- Establish the volume of subdrainage outflow for typical pavement systems in the field; and
- Define, based on quantitative laboratory and field data, the core flow capacity requirements for geocomposite fin-drain materials used in pavement edge drainage systems.

LABORATORY TESTING PROGRAM AND DATA

Laboratory Testing Equipment

The inlet end of the channel used for testing the core flow capacities of selected fin-drain materials is shown in Figure 1. The channel view from the downstream end is shown in Figure 2. The main equipment components used in the testing program consist of the flow channel which contains the fin-drain material and a weir box for measuring the volume of water flow. A schematic diagram of the laboratory testing equipment is shown in Figure 3.

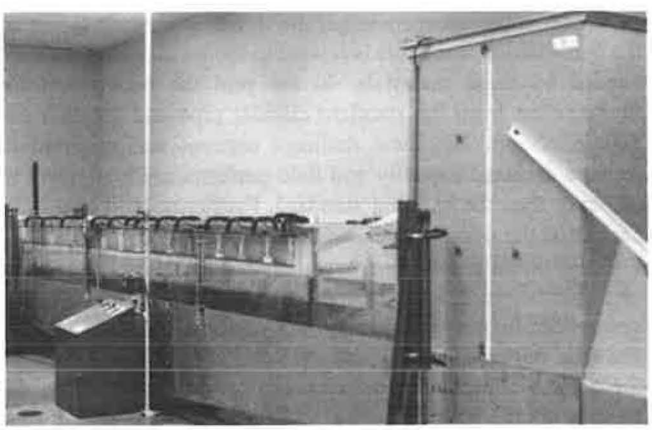

FIGURE 1 View of flow channel and a test section of fin-drain material from inlet end.

The testing equipment is located in the Hydrosystems Instruction Laboratory at the University of Illinois. The Plexiglas™ channel has a usable length of 24 ft, width of 1.5 ft, and depth of 2 ft. Channel slope can be varied from −5 to +15 percent through the use of hydraulically operated cylinders.

Flow to the channel is supplied from a constant head tank, which has a crest elevation of approximately 53 ft above the laboratory floor. The flow passes through a series of supply lines into an 8-in.-diameter pipe which empties into the 5-ft-high by 4-ft-long by 1.5-ft-wide head tank of the tilting channel. A series of baffles is located within the head tank as well as at the entrance of the channel to dampen the turbulence of the approach flow. The flow rate is regulated by a butterfly valve system installed in the 8-in. supply line.

FIGURE 2 View of flow channel and a test section of fin-drain material from downstream end.

Flow leaving the channel passes through an exit chute and into the weir box (Figure 4). The exit chute separates the measured flow in the fin-drain material from the bypass flow. The weir box is perpendicular to the end of the channel and empties into an underground sump. Water within the sump is pumped by a vertical turbine pump back into the main head tank where it reenters the water supply system.

The flow rate was measured through the use of a 20 degree sharp-crested V-notch weir plate attached to the 6-ft-long weir box located beneath the exit chute at the downstream end of the channel (Figure 4). The head above the crest of the weir was measured with a point gauge situated in a stilling well attached to the side of the weir box. The weir box was designed to accurately measure flows to within 0.5 gal/hr.

Laboratory Testing Procedure

The laboratory testing procedure was developed to evaluate the core flow capacity of nominal 12-in. geocomposite fin-drain sections 20 ft long. The geocomposite fin-drain material was sandwiched between one side of the Plexiglas flow channel and a plywood plate. The fin drain was firmly placed so that no flow occurred between the fin exterior and the wall restraints. The top of the fin drain was also sealed so that, even under a submerged entrance head, all flow would be confined to the fin-drain core. Water level at the entrance of the fin-drain material was controlled by the pipe inlet valve and by a small spillway (Figure 5), which diverted excess water to the channel flow area behind the braced plywood backing plate. By use of different spillway heights, core flow capacities at entrance head levels of approximately 6 in., 12 in., and 18 in. were evaluated during the testing program. Water that passed over the spillway was diverted away from the weir box at the outlet end by use of the baffles in the exit chute (Figure 4). Flow measurements were conducted at 0, 1, 2, 3, and 4 percent channel slopes.

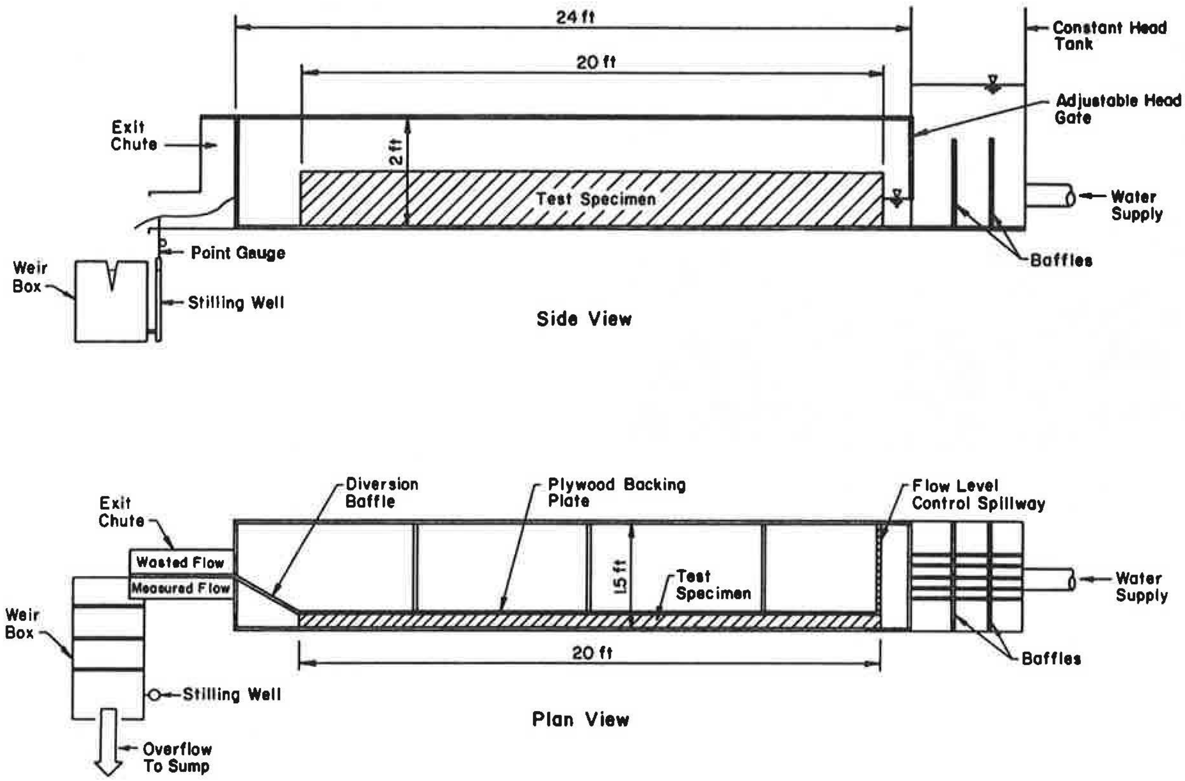

FIGURE 3 Schematic plan and side view of laboratory testing equipment.

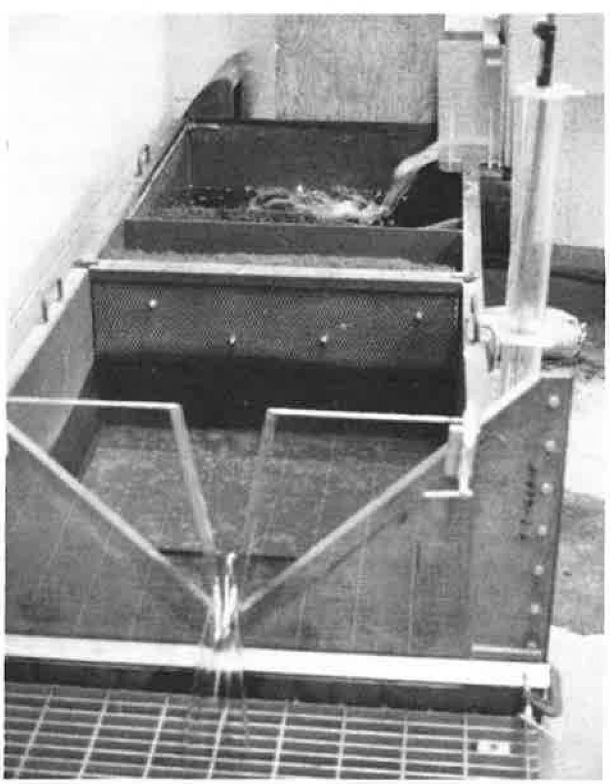

FIGURE 4 Weir box with 20 degree sharp-crested V-notch weir plate.

The quantity of flow measured at the 20 degree sharp-crested V-notch weir was computed by use of the following equations (*5*):

$$Q = 8/15\ C_d \tan \theta/2\ [2g(H)^5]^{1/2} \qquad (1)$$

$$H = h + 0.0095 \qquad (2)$$

where

Q = flow volume, ft^3/s,
C_d = weir coefficient of 0.593,
θ = V-notch weir angle, degrees,
g = gravity term, 32.2 ft/s^2, and
h = depth of flow in the V-notch weir, ft.

Based on catch sample volume measurements at the weir, Equation 1 was found to provide flow volume predictions that compared favorably with the catch samples throughout the range of flows evaluated in the testing program.

Fin-Drain Materials Tested

A description of the core and fabric wrap of the selected geocomposite fin-drain materials evaluated in this testing program is provided in Table 2. All of the fin-drain materials were

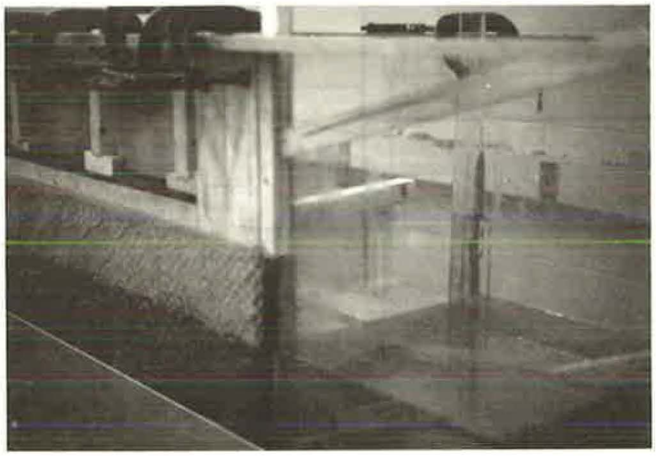

FIGURE 5 Entrance head control in a fin-drain material by using a 6-in. spillway height.

TABLE 3 LABORATORY FLOW VOLUMES AS A FUNCTION OF CHANNEL SLOPE AND ENTRANCE HEAD FOR 20-FT-LONG SECTIONS OF GEOCOMPOSITE FIN-DRAIN MATERIALS

		Flow Volume (gal/hr) for Channel Slope Percentage				
Fin-Drain Material	Entrance Head (in.)	0	1	2	3	4
A[a]	6.2	110	158	189	223	249
B[a]	6.3	387	550	670	782	892
C[a]	6.3	98	133	154	189	215
D	6.3	45	67	79	93	108
E	6.1	270	357	407	495	564
F	6.4	21	30	39	47	55
A[a]	12.2	305	380	435	495	536
B[a]	12.3	1,065	1,281	1,444	1,601	1,787
C[a]	12.5	273	336	380	423	468
D	12.2	147	170	191	220	237
E	12.4	655	794	892	990	1,080
F	12.5	66	92	106	114	133
A[a]	18.2[b]	517	598	660	703	753
B[a]	18.5[b]	1,692	1,875	2,026	2,163	2,284
C[a]	18.5[b]	443	490	541	564	584
D	18.4[b]	218	252	273	295	318
E	18.3[b]	997	1,137	1,235	1,350	1,390
F	18.5[b]	123	141	153	165	178

NOTE: All test sections 12 in. nominal depth; see Table 2 for material description.
[a]Two-sided flow.
[b]Submerged entrance.

tested as supplied by the manufacturer and all test sections were 20 ft long with a nominal depth of 12 in.

Test Data

The core flow capacities of the geocomposite fin-drain materials as a function of channel slope and entrance head are presented in Table 3. The average entrance heads were 6.3 in., 12.3 in., and 18.4 in. The flow volumes for fin-drain materials A, B, and C represent total two-sided flow since these materials have a centrally located impermeable core membrane. Products D, E, and F use open core systems that do not divide flow.

FIELD SUBDRAINAGE OUTFLOW STUDIES

Typical subdrainage outflow with time relationships, developed from a previous research project, for several precipitation events on I-57 near Champaign, Illinois, during 1978 are shown in Figures 6 through 8 (6).

Figures 6 and 7 show outflow for a continuously reinforced pavement with unsealed and sealed pavement edge-shoulder joints, respectively. An outflow relationship for a conventional reinforced, jointed concrete pavement with 100-ft joint spacings is shown in Figure 8. Both the jointed and continuously reinforced pavement test sections had longitudinal slopes less than 1 percent. The pavement edge drainage systems used are shown in Figure 9. Flow measurements were conducted at outlets spaced at 500-ft intervals. Outflow was measured at the test site by using a tipping-bucket flow meter (6).

Typical subdrainage outflow with time relationships obtained from I-80 near Morris, Illinois, during 1983 and 1984

TABLE 2 DESCRIPTION OF GEOCOMPOSITE FIN-DRAIN MATERIALS TESTED

Core Data				Fabric Data		
Fin-Drain Material	Structure	Material	Thickness (in.)	Material	Fabrication	Core Attachment
A	Cuspated	HDPE[a]	0.78	Polypropylene	Nonwoven	Loose wrapped
B	Cuspated	HDPE	1.57	Polypropylene	Nonwoven	Loose wrapped
C	Cuspated	HDPE	1.00	Polypropylene	Nonwoven	Adhesive bond one side, loose one side
D	Dimpled sheet	HIPS[b]	0.38	Polypropylene	Nonwoven, needle punched	Adhesive bond two sides
E	Columns	LLDPE[c]	1.00	Polypropylene	Nonwoven, needle punched, calendered	Adhesive bond to columns, heat bond backing
F	Net	LDPE[d]	0.25	Polypropylene	Nonwoven	Linear adhesive bond line both sides

[a]HDPE = High-density polyethylene.
[b]HIPS = High-impact polystyrene.
[c]LLDPE = Linear low-density polyethylene.
[d]LDPE = Low-density polyethylene.

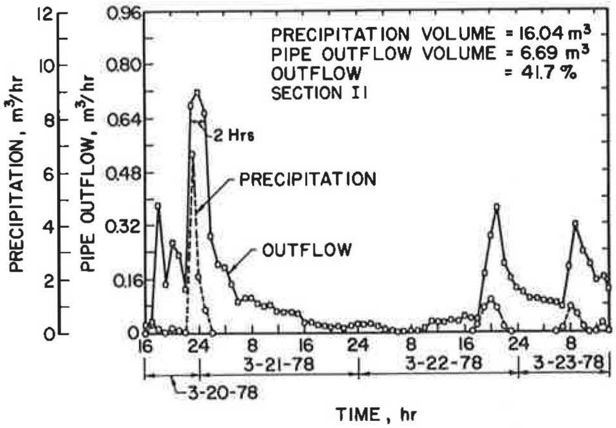

FIGURE 6 Influence of precipitation on subdrainage outflow in a continuously reinforced concrete pavement section without a sealed edge joint (*6*).

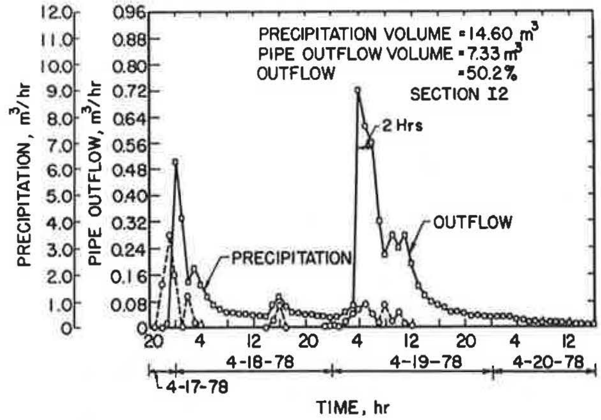

FIGURE 8 Influence of precipitation on subdrainage outflow in a jointed concrete pavement section without a sealed edge joint (*6*).

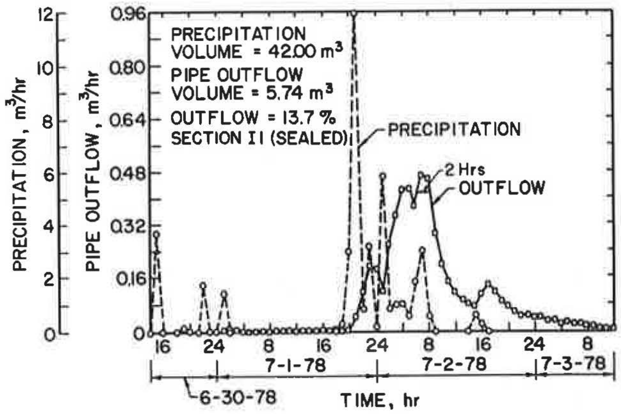

FIGURE 7 Influence of precipitation on subdrainage outflow in a continuously reinforced concrete pavement section with a sealed edge joint (*6*).

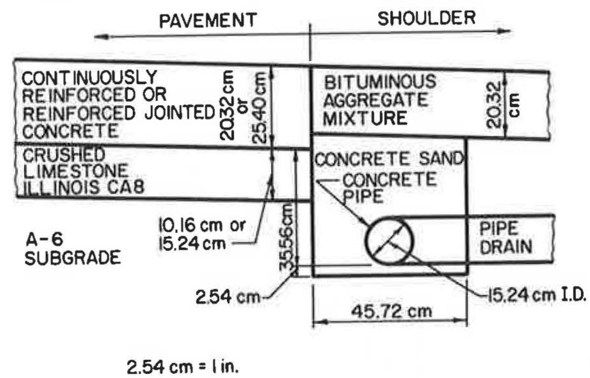

FIGURE 9 Subdrainage systems at test sections on I-57 in Illinois.

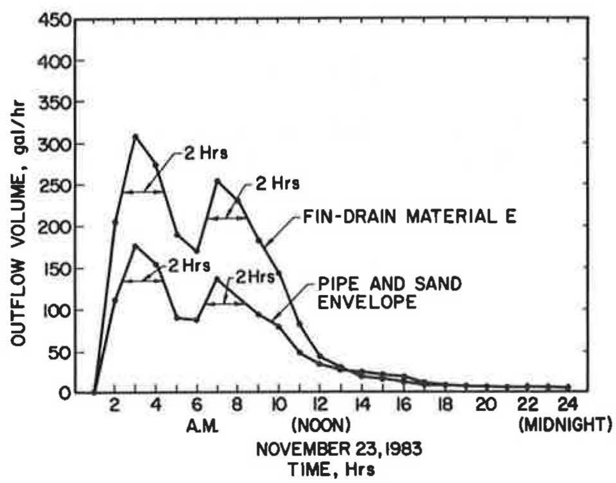

FIGURE 10 Subdrainage outflow volumes from I-80 near Morris, Illinois, for precipitation event on November 23, 1983.

are shown in Figures 10 through 13.. This study was conducted by the Illinois Department of Transportation (IDOT) to compare outflow between a standard 4-in.-diameter polyethylene pipe and sand envelope system and an 18-in.-deep geocomposite fin-drainage system (fin drain E in Table 2). The longitudinal slopes of the two pavement test sections were less than 1 percent. An outlet spacing of 500 ft was used for both drainage test sections. Tipping-bucket flow meters were used to measure the outflow volumes (*6*).

ANALYSIS AND DISCUSSION OF LABORATORY AND FIELD STUDIES

Laboratory Results

As shown in Table 3, there is a broad range of core flow capacities for the various fin-drain materials presently on the market. It is also apparent that core flow capacity is dependent upon the core dimensions, core geometry, entrance head, and channel slope. Graphical relationships between the core flow capacities of the various fin-drain test sections and channel slopes for different entrance heads are shown in Figures 14 through 16. Figure 14 shows the core flow capacities for nominal 12-in.-deep fin-drain sections with a 6.3-in. entrance head (an entrance flow of about 1/2 of the nominal depth of the fin-drain section). It is interesting to note the range of differences between the core flow capacities of the materials tested. Fin drain B with a 1.57-in. core thickness displayed the highest flow capacity with channel slope. As expected, the core capacity of this product increased as the entrance head was increased as shown in Figures 15 and 16. The core flow capacities shown in Figure 15 for the average 12.3-in. entrance

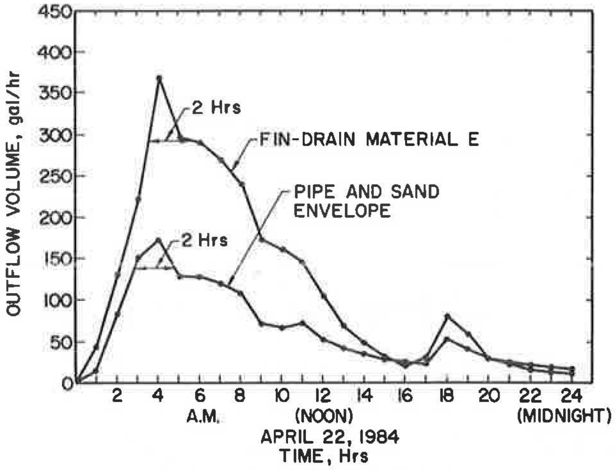

FIGURE 11 Subdrainage outflow volumes from I-80 near Morris, Illinois, for precipitation event on April 22, 1984.

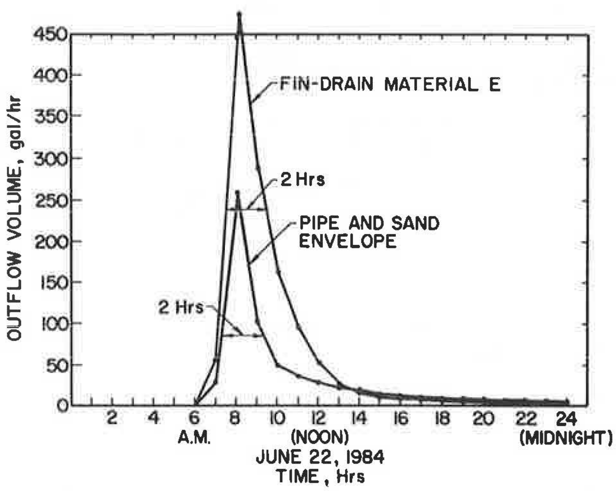

FIGURE 12 Subdrainage outflow volumes from I-80 near Morris, Illinois, for precipitation event on June 22, 1984.

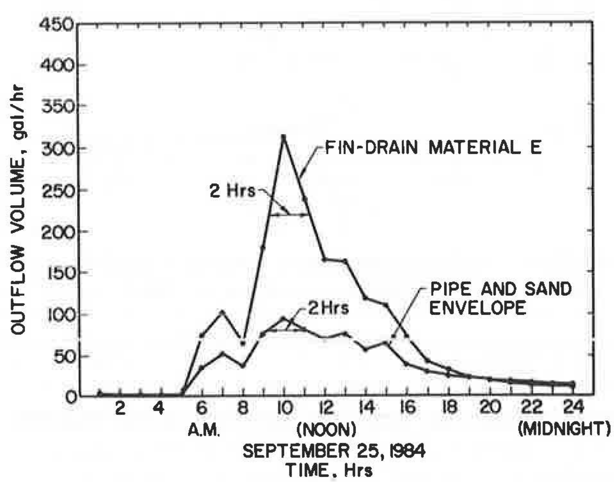

FIGURE 13 Subdrainage outflow volumes for I-80 near Morris, Illinois, for precipitation event on September 25, 1984.

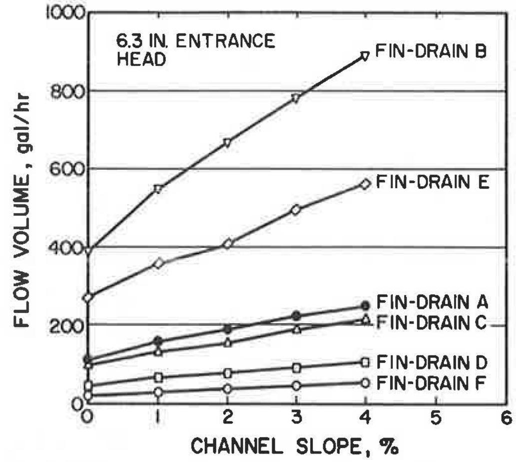

FIGURE 14 Relationships between core flow capacity and channel slope at 6.3-in. average entrance head.

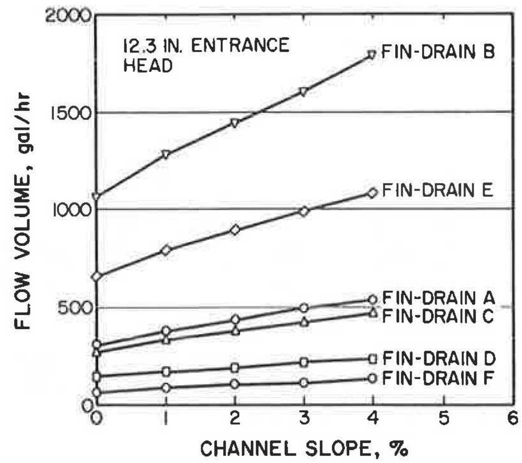

FIGURE 15 Relationships between core flow capacity and channel slope at 12.3-in. average entrance head.

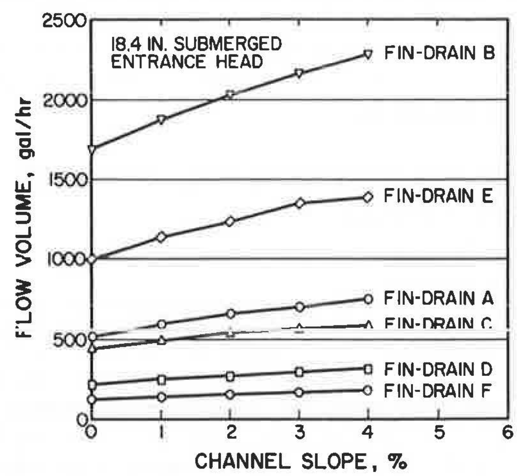

FIGURE 16 Relationship between core flow capacity and channel slope at a submerged 18.4-in. average entrance head.

head provide data for full flow in nominal 12-in.-deep fin-drain sections. The submerged head flow capacities shown in Figure 16 are of general interest to the evaluation of core geometry on flow capacity and would not be the normal situation for pavement edge drainage.

Fin drains C and E have similar dimensions of 1 in. wide by 12 in. deep (Table 2). Figures 14 through 16 show that the core flow capacities of these two products are substantially different throughout the range of channel slopes evaluated. The fin-drain E core capacity ranges from 1.4 to 1.7 times greater than the fin-drain C core capacity regardless of entrance head and channel slope. It is evident that core geometry has a major influence on the flow capacities of these two products which have similar outside dimensions. In fact, in Figures 14 through 16 it is shown that fin drain A with a 0.78-in. core thickness provided greater flow capacity than the 1-in. fin-drain C core, regardless of entrance head or channel slope.

Except for the fin-drain C material, the core flow capacities of the fin-drain materials tested increased relative to the core thickness. Although flow volume is related to core thickness, there is not a proportional relationship between fin drains with different core geometries (Figures 14 through 16).

Relationships Between Laboratory Results and Field Requirements

A sketch of a typical pavement structural subdrainage system using a fin drain is shown in Figure 17. An important consideration when choosing a fin-drain system or pipe envelope system is that adequate trench depth and width are provided to ensure that the water does not back up into the pavement structural layers while being carried to the outlet. Since the fin drain functions as both a collector and a conduit it needs proper dimensions (thickness and width), flow capacity, and outlet spacing to maintain the water level in the fin core at a depth below the pavement structural layers a majority of the time. In Figure 17 this flow should be restricted to that portion of the fin below the subbase-subgrade interface or "freeboard" area. Based on Table 1 the core flow capacity in the "flow zone" below the freeboard area should be such that water will not be retained in the structural pavement section for more than 2 hr for excellent drainage nor more than 1 day for good drainage.

In referring to Figures 6 through 13 it becomes apparent that

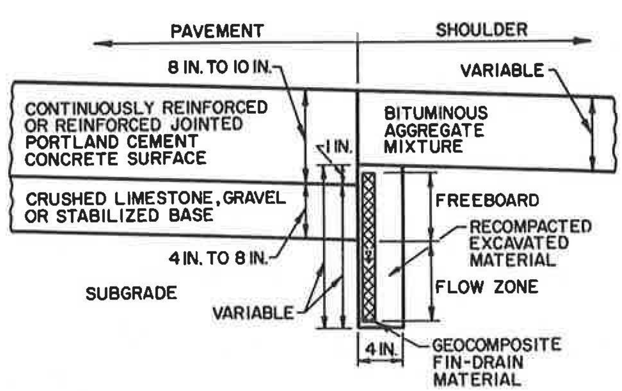

FIGURE 17 Typical geocomposite fin-drain system showing freeboard and flow zone areas.

the Illinois standard subdrain system with a sand envelope can display 2-hr periods with outflow volumes ranging from approximately 70 gal/hr to over 150 gal/hr. The fin-drain system (fin drain E in Table 2) in Figures 10 through 13 shows even higher 2-hr-period outflow volumes which range from approximately 220 gal/hr to almost 300 gal/hr. It is probable that a pipe subdrainage system with a coarse aggregate envelope would provide outflow volumes similar to this fin-drain system.

From the observed outflows for the Illinois standard pipe drain system and for the fin-drain system it would appear that the flow zone capacity of a fin-drain core should be in excess of 150 gal/hr to match a standard pipe and sand envelope system and in excess of 300 gal/hr to match a quality geocomposite, fin-drain system in order to provide excellent drainage performance (drainage of structural pavement section in 2 hr or less).

In the past it has been a common practice to construct the fin-drain system with at least 4 to 6 in. of the drain extending into the subgrade below the subbase-subgrade interface or flow zone area in order to function as a conduit to carry water to the outlet. In referring to Table 3 and Figures 14 through 16 it can be seen that only fin drain B with a flow capacity of 387 gal/hr (0 percent slope) to 892 gal/hr (4 percent slope) and fin drain E with a flow capacity of 270 gal/hr (0 percent slope) to 564 gal/hr (4 percent slope) would qualify as excellent subdrainage systems under the new AASHTO Guide criteria (Table 1) if a 6-in. flow zone is desired. This is not to say that the other fin drains cannot be used, however. The flow capacities of fin drains A and C could be improved by increasing their overall depth dimension to provide a flow zone depth of approximately 12 in. or possibly by decreasing outlet spacing. It is important to note that the centrally located impermeable core used by fin drains A, B, and C may be restrictive, and total core flow capacity may be less than that shown in Table 3. Furthermore, the flow capacities shown in Table 3 were measured for conditions of no fabric sag into the core. Fin-drain materials in which the fabric is loose-wrapped around the entire core or a portion of the core should be used with the understanding that actual field flow capacities may be considerably less because of excess fabric sag into the core. In fact, both laboratory and field observations made during this study indicated that fin-drain materials using a loose-wrapped fabric would have a high probability of diminished core flow capacity because of fabric sag into the core. It is felt that those fin-drain materials with the fabric bonded to the core are less likely to experience detrimental fabric sag into the core during construction operations and during their performance life.

Core flow capacity and flow efficiency are being found important to pavement performance. By quickly removing water from the structural pavement section and not allowing the water which seeps into the pavement edge shoulder joint to flow into the structural base or subbase sections, it is felt that pavement performance can be improved. Both joint faulting and transverse crack faulting on the outside lanes of I-80 near Morris, Illinois, are shown in Figures 18 and 19. The westbound lane is drained by fin drain E described in Table 2. The eastbound lane is drained using the standard Illinois subdrainage system composed of a 4-in. fabric wrapped, perforated, polyethylene pipe with a sand envelope. Three 1,000-ft test sections were measured in each of the two directions. The pavement had been ground smooth in the summer of 1983 when the subdrainage systems were installed. As shown in both Figures 18 and 19 the pavement section with the fin subdrainage system is experiencing considerably less joint and crack faulting than that using the standard system. Average joint faulting after 4 yr (1987) is about 60 percent less and the average crack faulting is about 30 percent less for the fin-drain system as compared to the standard. Traffic data from I-80 near Morris, Illinois, showed that the traffic volume ranged from about 1.3 million 18 thousand single-axle loads (SALs) in 1983 to about 1.7 million 18 thousand SALs in 1987 in the outer lane for each traffic direction. Based on the fact that both westbound and eastbound traffic on I-80 are similar in volume and weight distribution it would appear from Figures 18 and 19 that improved drainage capacity and efficiency provided substantial decreases in joint and transverse crack faulting during the 4 years of pavement service after the surface was ground smooth.

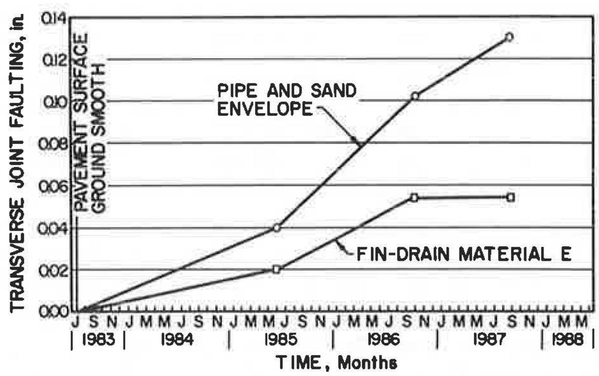

FIGURE 18 Influence of subdrainage type on transverse joint faulting on I-80 near Morris, Illinois.

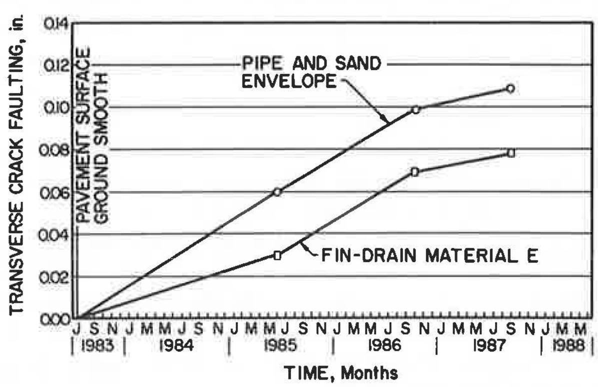

FIGURE 19 Influence of subdrainage type on transverse crack faulting on I-80 near Morris, Illinois.

CONCLUSIONS

Based on field and laboratory evaluations it is felt that any new drainage materials or systems should be comparable to or better than existing systems in terms of core flow capacity and flow efficiency. In fact, results from field evaluation of the fin-drain system at the Morris, Illinois, test site would indicate that the standard pipe envelope underdrain system should be improved to provide for better pavement performance. This can be accomplished by replacing the less permeable sand envelope with a more permeable coarse granular envelope material.

As for the fin drain, it is important that adequate flow below the freeboard area be maintained so that during periods of drainage the fin core will perform as a sink for water and not as a source of water to the structural pavement section. Based on subdrain outflow data obtained to date it would appear that a fin drain should have a flow zone capacity that will ensure that water flow in the fin section itself will not be in contact with structural pavement components for a period exceeding 2 hr. This study indicates that geocomposite fin-drain systems will be required to provide flow zone capacities in excess of 150 gal/hr at 0 percent pavement gradient and in excess of 200 gal/hr at gradients of 1 percent or greater to compare with a standard pipe and sand envelope system. When compared with a more permeable aggregate envelope system or a high-performance fin-drain system, flow zone capacities in excess of 200 gal/hr to 300 gal/hr may be desirable depending upon the pavement gradients and the number and size of joint and crack openings. Faulting data shown in Figures 18 and 19 would indicate performance advantages in designing a subdrainage system toward the higher values of core flow capacity.

When selecting a geocomposite fin-drain material for subdrainage applications, it is important to ensure that its structural properties meet design specifications in addition to meeting flow volume requirements. It is further recommended that fin-drain materials that use a loose-wrapped fabric not bonded to the core projections be used with caution since there is a high probability of fabric sag into the core and subsequent decrease in drainage efficiency.

Any new fin-drain material should be carefully evaluated to ensure that its projected core flow capacity and drainage efficiency will be equivalent to or exceed present systems. There still remain too many unknowns in the drainage area to not select subdrainage systems that are conservative or have a factor of safety in favor of the design engineer and good pavement performance.

In time it is felt that even better fin-drain systems and pipe envelope systems will be developed which will improve pavement performance. Until these are developed, it is important that pavement drainage not be compromised. Fin-drain materials for pavement edge drainage systems should be selected based on performance attributes as well as material and construction costs.

ACKNOWLEDGMENTS

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REFERENCES

1. *AASHTO Guide for Design of Pavement Structures*. NCHRP Project 20-7/24. National Cooperative Highway Research Program and AASHTO Joint Task Force on Pavements, Washington, D.C., 1985.
2. B. J. Dempsey. Laboratory and Field Studies of Channeling and Pumping. In *Transportation Research Record 849*, TRB, National Research Council, Washington, D.C., 1982, pp. 1–12.
3. J. H. Woodstrom. Concrete Pavement Drainage in California. Presented at International Seminar on Drainage and Erodibility at the Concrete Slab-Subbase-Shoulder Interfaces, Laboratoire Central des Ponts et Chaussées, Paris, France, 1983.

4. M. I. Darter, J. M. Becker, M. B. Snyder, and R. E. Smith. *NCHRP Report 277: Portland Cement Concrete Pavement Evaluation System—COPES.* TRB, National Research Council, Washington, D.C., 1985, p. 175.
5. I. H. Shames. *Mechanics of Fluids.* McGraw Hill, Inc., New York, 1982.
6. B. J. Dempsey and Q. L. Robnett. Influence of Precipitation, Joints, and Sealing on Pavement Drainage. In *Transportation Research Record 705,* TRB, National Research Council, Washington, D.C., 1979, pp. 13–23.

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Hydraway Edgedrain Experience in Ohio

KEITH T. HINSHAW

The purpose of this paper is to compare the effectiveness of the Hydraway™ edgedrain, which is manufactured by the Monsanto Company, with the Ohio Department of Transportation's standard 4-in. pipe underdrain without fabric wrap. The 650-ft-long test sections are contiguous, constructed at the same grade, and have separate outlets. The U.S. Geological Survey, Water Resources Division, installed equipment to measure the discharge from each test section. Preliminary results from the monitoring devices are not conclusive. There have been too many gaps in the data. For various reasons, such as damage from lightning, an automobile accident, and at times water backing up in the ditch, complete data for the most significant rainfall events are not available. Neither system has shown consistently superior performance over the other. The costs for the Hydraway on the two projects in 1985 were $4.10 and $5.50/ft. The costs for three projects in 1986 and 1987 were $2.42, $2.80, and $2.85/ft. This compares to an average cost of $2.50 to $3.00/ft for the standard pipe underdrain. Collection of data will be continued until matching data for the two drainage systems can be compared and definite conclusions made.

The removal of water from roadway subbase and subgrade is an important factor in extending the life of a pavement. Ohio, like many other states, has experienced numerous drainage-related pavement problems. Therefore, when the Ohio Department of Transportation was approached by the Monsanto Company with an innovative concept for draining its pavements, considerable interest was expressed. The Hydraway edgedrain, consisting of a polyethylene core wrapped with filter fabric (Figure 1), was developed through research conducted for Monsanto by Barry J. Dempsey, Professor, Department of Civil Engineering, University of Illinois.

INSTALLATION

In August, 1985, 23,500 lineal ft of prefabricated edgedrain was installed on I-70 near SR 37, in Licking County, approximately 16 mi east of Columbus. The project involved complete rehabilitation of the eastbound two-lane pavement, including milling off the existing asphalt overlay (4$^1/_2$ in.), cracking and seating the existing concrete pavement, and overlaying with 9 in. of asphalt concrete.

The prefabricated edgedrain was installed at a depth of 33 in. adjacent to the outside edge of pavement, for the entire length of the project, with the exception of a 650-ft control test section, on which the Ohio Department of Transportation (ODOT) standard 4-in.-diameter shallow pipe underdrain was

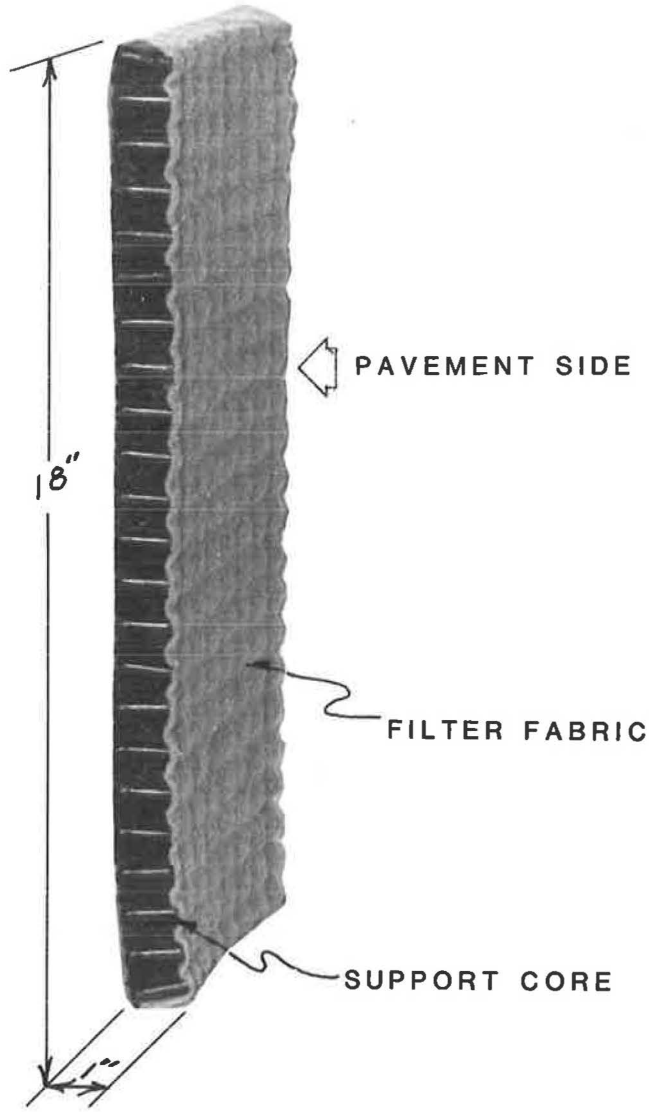

FIGURE 1 Edgedrain details.

used (Figure 2). The prefabricated drain was placed so that its top edge extended 3 in. above the bottom of the existing 9-in. portland cement concrete (PCC) pavement, as shown in Figure 2. The standard shallow pipe underdrain consisted of a 4-in. perforated polyethylene pipe, placed to a depth of 54 in. below the top of the pavement in a 12-in.-wide trench, which was backfilled with No. 8 aggregate (Table 1). Fabric wrap of the trench was not provided. Specifications of the edgedrain fabric are as follows: Specification 712.09, Filter Fabric. The fabric

Ohio Department of Transportation, 25 South Front Street, Columbus, Ohio 43215.

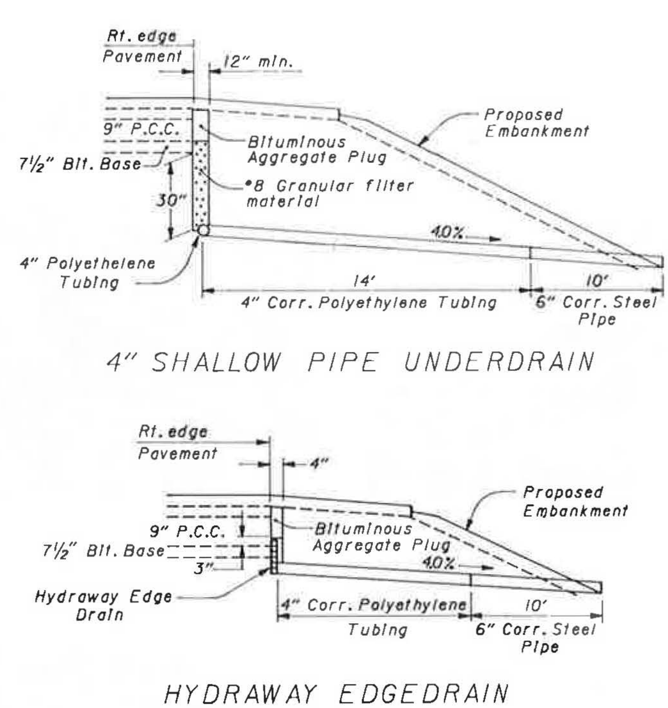

FIGURE 2 Typical sections.

TABLE 1 TRENCH BACKFILL SPECIFICATIONS—SIZES OF COARSE AGGREGATE (AASHTO M43)

Size Number	Nominal size square openings	Amounts finer than each laboratory sieve (square openings) percentage by weight								
		1	3/4	1/2	3/8	No. 4	No. 8	No. 16	No. 50	No. 100
7	1/2 to No. 4	....	100	90 to 100	40 to 70	0 to 15	0 to 5	....	......	....
78	1/2 to No. 8	....	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	......	....
8	3/8 to No. 8	....	....	100	85 to 100	10 to 30	0 to 10	0 to 5	......	....
89	3/8 to No. 16	....	....	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	....
9	No. 4 to No. 16	....	....	....	100	85 to 100	10 to 40	0 to 10	0 to 5	....
10	No. 4 to 0 (2)	....	....	....	100	85 to 100	....	....	......	10 to 30

shall be composed of strong rot-proof polymeric fibers formed into a woven or nonwoven fabric that meets the following requirements:

- Minimum tensile strength—80 lb
- Minimum puncture strength—25 psi
- Minimum tear strength—25 lb
- Minimum burst strength—130 psi
- Equivalent opening size: Soil Type 1 (soils with 50 percent or less passing U.S. No. 200 sieve)—EOS ≤ 0.6 mm; Soil Type 2 (soils with 50 percent to 85 percent passing U.S. No. 200 sieve)—EOS ≤ 0.3 mm
- Permeability—1×10^{-2} cm/sec

The installation of the prefabricated edgedrain was continuous, with a Vermeer trencher used to cut the 4-in.-wide trench in which it was placed. The edgedrain was placed immediately after the trench was cut by the use of an outrigger and a boot. The trench was then backfilled in two lifts, using the previously excavated material, which consisted mostly of granular subbase (Figures 3–5). Specifications of the subbase are given in Table 2. A small vibrating compactor, which was attached to the boot with a chain, completed the installation process.

The outlets were installed separately after all of the edgedrain was in place. The outlets consisted of a section of 4-in. corrugated polyethylene tubing which connected to the Hydraway end section. The tubing was then connected into a 6-in.-diameter, 10-ft-long corrugated steel pipe. The contractors have an option when installing animal guards. They may drill the end of the pipe and install the bars (Figure 6) or bolt on a metal collar (Figure 7). On this project, the collar was used. As a result of mowing operations, most of the collars have been knocked off and approximately 50 percent of the steel pipes have bent; however, the outlets are still functioning.

A minor problem developed the first day while cutting the trench. The existing pavement had been patched extensively,

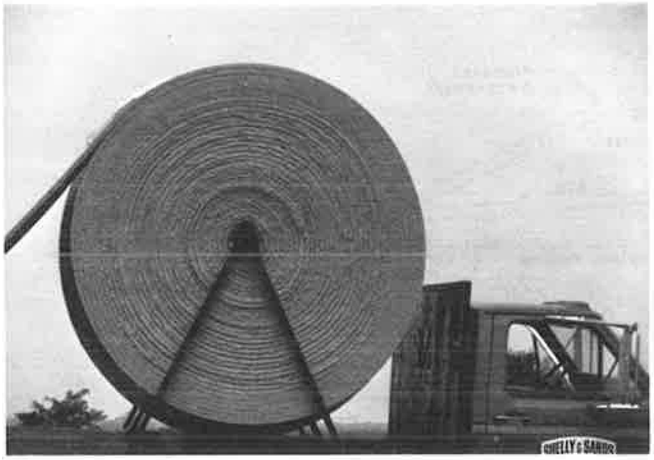

FIGURE 3 Roll of edgedrain.

FIGURE 4 Before installation.

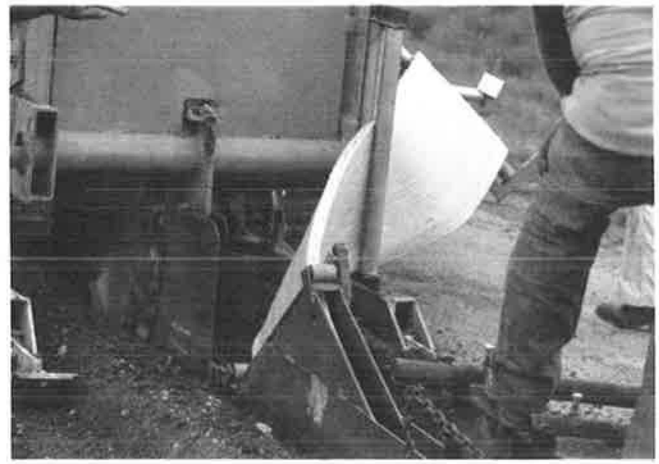

FIGURE 5 Edgedrain entering the boot.

FIGURE 6 Animal guard.

TABLE 2 SUBBASE SPECIFICATIONS

Total Percent Passing		
Sieve	Grading A	Grading B
2 1/2 inch	100	100
1 inch	70-100	70-100
No. 4	25-100	25-100
No. 40	5-50	10-50
No. 200	0-10	5-15

NOTE: Specification 310.02. Materials. Materials furnished under this item shall be gravel, crushed slag, crushed stone, sand, granulated slag, a mixture of crushed and granulated slags, or other types of suitable materials meeting the requirements of this item and having the approval of the director. The sodium sulfate soundness loss for aggregates shall not exceed 15 percent. However, where the major portion of the unsound material in a coarse aggregate acquires a mud-like condition when tested for soundness, the maximum loss shall be 5 percent for all uses. In addition, open-hearth and basic-oxygen furnace slag shall conform to stockpiling and aging requirements of 703.01.

and in many locations, aggregate drains had been constructed adjacent to the patches. The presence of these drains, the badly deteriorated pavement, and the 7-in.-thick asphalt shoulder, made it difficult to keep the trencher properly aligned. Because of this, there were a few times when reinforcing steel was pulled out from the edge of pavement. A second trencher, which had a wider cut of approximately 6-in., was used to cut through the asphalt shoulder. Some of the subbase in the area of

FIGURE 7 Animal guard (collar style).

FIGURE 8 Tipping bucket enclosure.

FIGURE 9 Tipping bucket.

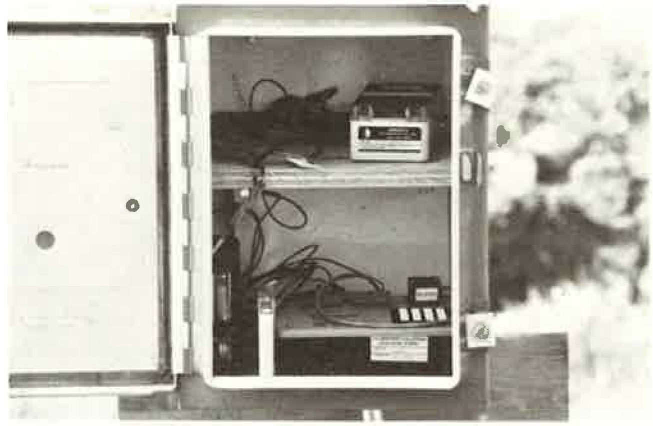

FIGURE 10 Data collection equipment.

FIGURE 11 Drilling into the edgedrain.

FIGURE 12 Viewing pipe.

FIGURE 13 Borescope.

FIGURE 14 Digging up an outlet.

the patches fell into the trench, a situation which was unavoidable. The installation rate varied from 20 to 50 ft/min. On future projects of this type, the sequence of construction will specify that planing of the asphalt overlay be done first to allow the trencher operator to locate the edge of the existing concrete pavement.

In August, 1985, 9,800 lineal ft of prefabricated edgedrain was installed on a four-lane divided pavement with a curbed median, on US-36 near Newcomerstown, Ohio. The edgedrain was placed on the outside edge of pavement for a distance of 4,900 ft, through a shale cut section. The existing pavement was in extremely poor condition. Patching and undersealing were done before resurfacing with 3 in. of asphalt.

FIGURE 15 Outlet pipe removed.

In the summer of 1986, 88,000 lineal ft of prefabricated edgedrain was installed on US-30 near VanWert, Ohio. The edgedrains were placed on both sides of this four-lane divided pavement. The original pavement was a composite design with concrete base. The joints were still in good condition; therefore, the rehabilitation was minor, consisting of asphalt undersealing and some shoulder and bridge approach repairs. The overlay was 2 in. of asphaltic concrete with sawed joints located over the existing joints. This area of Ohio has flat topography with shallow ditches, which made it difficult to outlet the edgedrains, and necessitated the cleaning of many of the ditches. The outlets were approximately 500 ft apart and the plans indicated a straight grade between them; however, in order to outlet the edgedrains, they had to be laid to a grade which followed the actual pavement grade.

MONITORING

To verify the hydraulic performance of the prefabricated drain, ODOT contracted with the U.S. Geological Survey, Water Resources Division, Columbus, Ohio, to conduct discharge testing.

In April, 1986, the Water Resources Division installed equipment to measure the discharge from two contiguous 650-ft-long underdrain sections which were constructed at the same grade, each having separate outlets. The purpose of this installation was to measure the real-time discharge response of the standard shallow pipe underdrain and the prefabricated edgedrain systems. Tipping bucket gauges were installed at the outlets of the test sections (Figures 8 and 9). These gauges operate by causing a contact closure each time a preset volume

FIGURE 16 New outlet pipe.

of water passes through them. A microprocessor-controlled logger was used to record and total the contact closures in 10-min intervals (Figure 10). Consequently, volumes and flow rates could be determined. A third tipping bucket gauge was installed in the area to measure the rates and intensities of precipitation. An event recorder was used to store the precipitation data on an erasable, reprogrammable chip.

In June, 1987, Monsanto Company representatives installed ½-in. plastic inspection pipes at three locations along the 650-ft test section of their edgedrain on I-70 (Figures 11 and 12). The pipes were placed at each end and at the center of the 650-ft test section. Viewing of the inside of the edgedrain by use of a borescope is possible (Figure 13). In the upstream location there was no flow, but in the center and downstream locations there was approximately 6 in. of standing water, which was a concern because if there was an obstruction the flow data would be affected. A decision was made to dig up the outlet pipe in an attempt to determine the cause of the problem.

On July 14, 1987, with the assistance of an ODOT maintenance crew, the flexible polyethylene outlet pipe was dug up (Figures 14–16). There was a slight rise in the flexible pipe but not enough to be a major problem. It was suggested that a rigid outlet pipe would provide a straight slope without possibilities for variations. The existing pipe was removed, bottom of the trench regraded, and a new pipe installed. Before the new pipe was installed, a water hose, which was attached to a 500-gal tank, was connected at the upstream location. With only the pressure from the tank which was mounted on a flatbed truck, it took the water 35 min to flow the 640 ft (Figure 17).

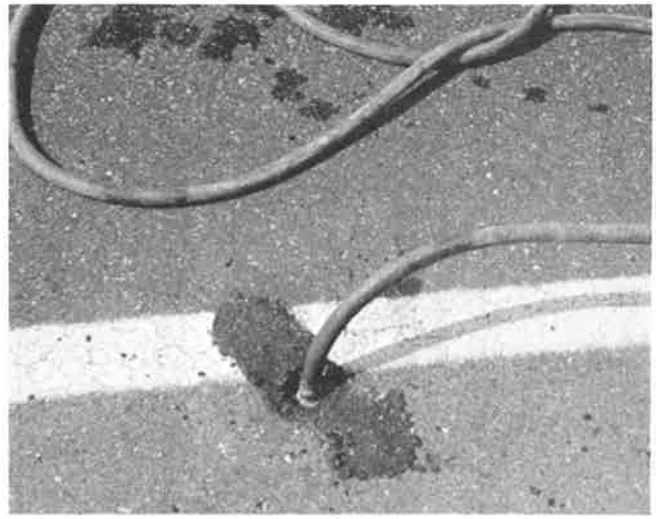

FIGURE 17 Flushing.

On July 15, 1987, the Monsanto representatives installed three borescope monitoring pipes on the eastbound lanes of the US-36 site. The location nearest the outlet pipe revealed clear water flowing to a depth of 1 in. The inside wall of the edgedrain had fines clinging to it and was functioning as predicted. The other two monitoring sites further upstream were relatively dry.

RESULTS

It cannot be determined from the average discharge charts (Appendix) which system is superior. There have been many gaps in the data due to lightning hits, an automobile accident, and at times water backing up in the ditch and rendering the tipping bucket inoperable. This has produced inconsistencies during some of the most significant rainfall events. Collection of data will continue until enough matching data for the two drainage systems can be compared and conclusions made.

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APPENDIX

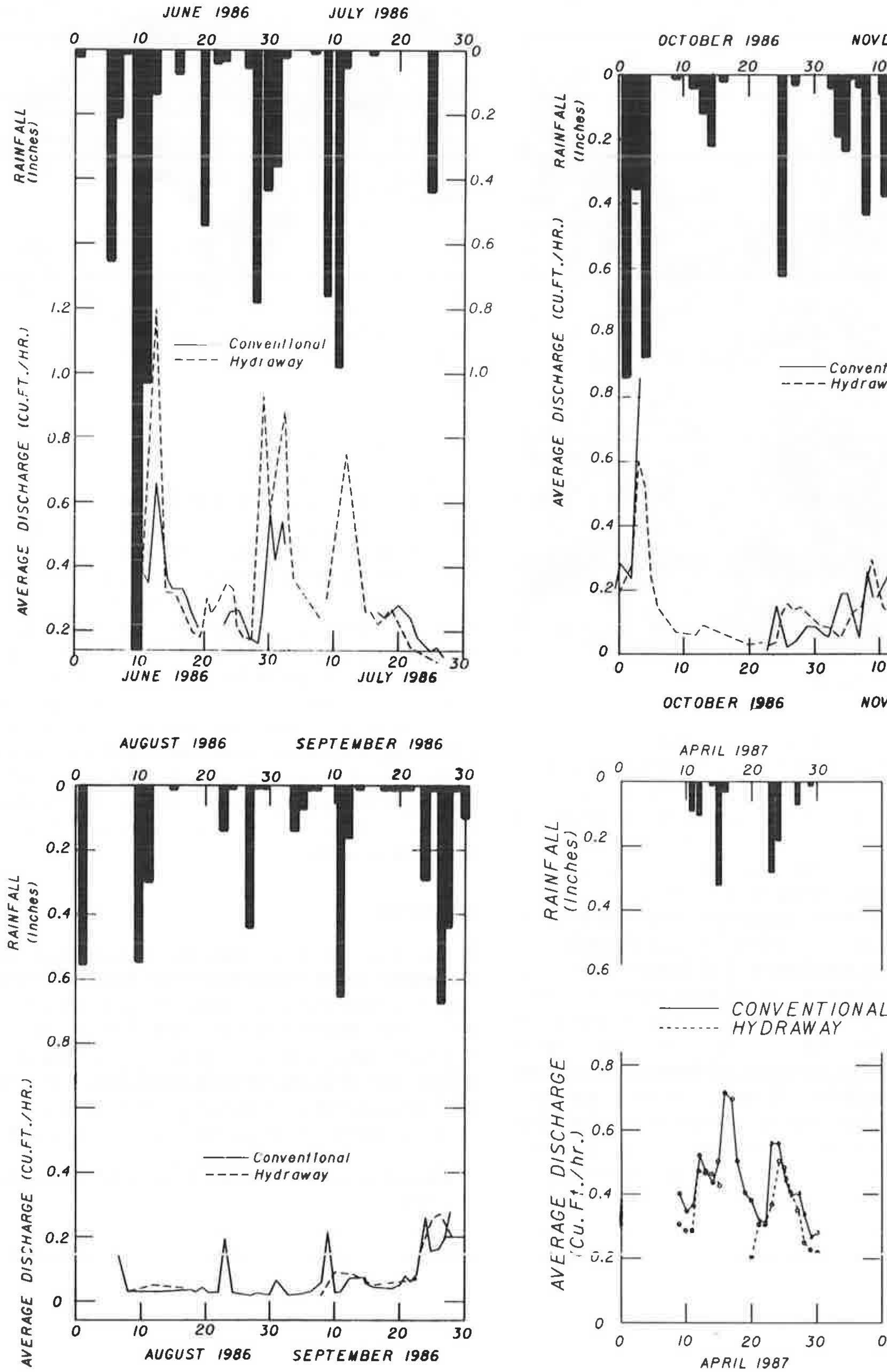

FIGURE A-1 Average discharge charts.

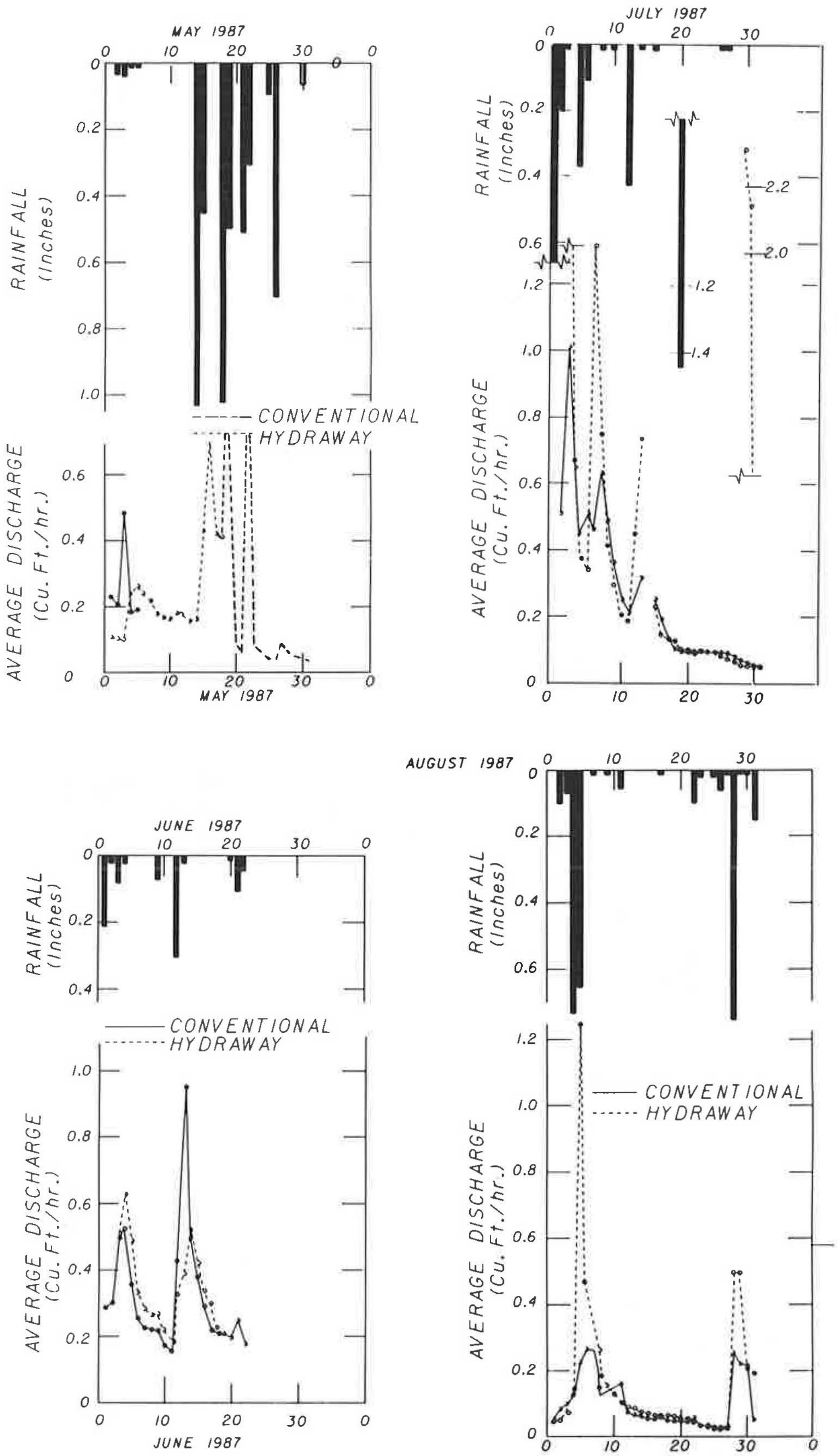

FIGURE A-1 *continued*

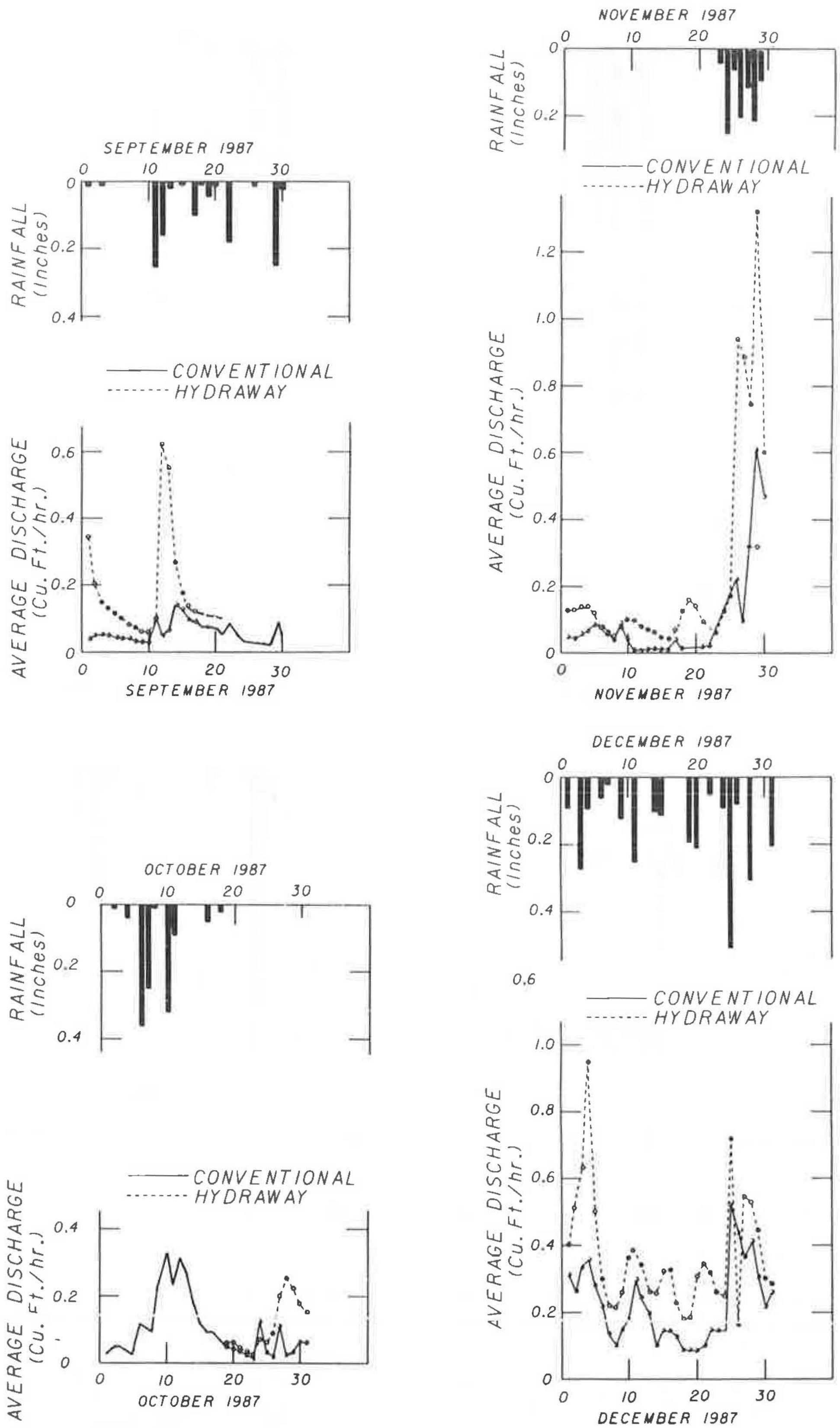

FIGURE A-1 *continued*

Performance of Wick Drains in New Orleans Clays

SUBAL K. SARKAR AND RAYMOND J. CASTELLI

Prefabricated wick drains and a preloading program were employed in the construction of a wharf facility in the city of New Orleans to accelerate consolidation of a thick layer of soft clayey soils. Preloading and wick drains were used for a portion of the wharf platform and for an adjoining 10-acre storage area. A proposed future storage area of approximately 32 acres was preloaded without the use of wick drains. A total of approximately 2 million linear feet of drains were installed for the project. Various instrumentations were monitored during construction to evaluate the performance of the preload program in areas both with and without wick drains. Consolidation parameters calculated from the field data are presented and compared with values used in the design of the wick drain installation. After a 12-mo preload period, the clayey soils in the wick drain area had reached an average of 80 percent consolidation in comparison to only about 28 percent consolidation in the nondrain area. The construction preload in the wick drain area successfully preconsolidated the clayey subsoils for the design load condition.

New Orleans, the second busiest port in the United States, has initiated the development of a major new facility on the Mississippi River–Gulf Outlet (MR-GO) within the city of New Orleans (Figure 1). This facility, called the Jourdan Road Terminal, will eventually include a total of ten berths, with related container and roll on/roll off terminals, and cargo handling areas.

The first stage in the development of the Jourdan Road Terminal was the completion, in 1983, of a 1400-ft-long wharf for Berths 4 and 5 (Figure 2). Construction for Berths 4 and 5 also included a 10-acre container storage area (Area A in Figure 2) adjoining the wharf platform, and a 32-acre storage area (Area B in Figure 2) proposed for future development. These two areas are separated by a U.S. Army Corps of Engineers flood protection levee. Area A was required to be in service at the completion of Berths 4 and 5. Development of Area B, however, would not be required until many years afterward.

The site of the Jourdan Road Terminal is underlain by approximately 60 ft of soft and highly compressible clays. During construction for Berths 4 and 5, various ground treatment techniques were employed to improve stability and limit postconstruction settlements of these soils. A portion of the wharf area was constructed using vibro-replacement stone columns and reinforced earth construction. In addition, preloading and wick drains were extensively used for a portion of the wharf area and for the 10-acre storage area (Area A) immediately behind the wharf. The 32-acre storage area (Area B) was preloaded without vertical drains since development of this area was to be delayed.

Parsons, Brinckerhoff, Quade & Douglas, Inc., 1 Penn Plaza, New York, N.Y. 10119.

The wick drains had an average length of 65 ft and were installed at a spacing of 5 ft. A total of approximately 2 million linear feet of wick drains were installed for this project. At the time of construction in 1981 this was the largest installation of wick drains in the United States.

The design and construction of the wharf were described by Castelli et al. (*1*) and Sarkar et al. (2) and are not addressed in this paper. This paper discusses the preloading program at Areas A and B, and includes a description of subsurface conditions, a summary of assumptions and criteria used for design of the wick drains, details of drain installation, and an analysis of the findings of a construction instrumentation monitoring program.

SUBSURFACE CONDITIONS

Geologic Setting

The project site lies within the Mississippi River Deltaic Plain, which consists of recent deltaic deposits overlying soils of Pleistocene age (*3*, *4*).

The Pleistocene soils are deltaic deposits formed by the ancestral Mississippi during interglacial periods. As the glaciers advanced, the lowered sea level exposed the Pleistocene deposits to subaerial weathering, desiccation, and erosion. Consequently, the Pleistocene deposits are typically composed of stiff clays and dense sands, which provide a favorable bearing layer for foundations.

As the Pleistocene ended and sea levels rose, recent deltaic deposition began. Recent deltaic deposits present at the project site can be classified as marine, fluvial-marine, and paludal. The marine deposits were formed in a bay-sound environment on the surface of the Pleistocene soils, and generally consist of shelly fine sand containing silt and clay. Marine deposition gradually built up the sea floor until active deltaic advance became possible. This stage included the deposition of prodelta clays at a distance from the mouth of the existing delta. The gradual reduction in river velocity with increasing distance from shore resulted in deposits grading from silty clays near the river mouth to more plastic clays further from shore. Intradelta silts and fine sands were deposited by the distributary channels which built out over the prodelta deposits. Interdistributary deposits of clay with lenses of silt and fine sand then filled the depressions between distributary channels.

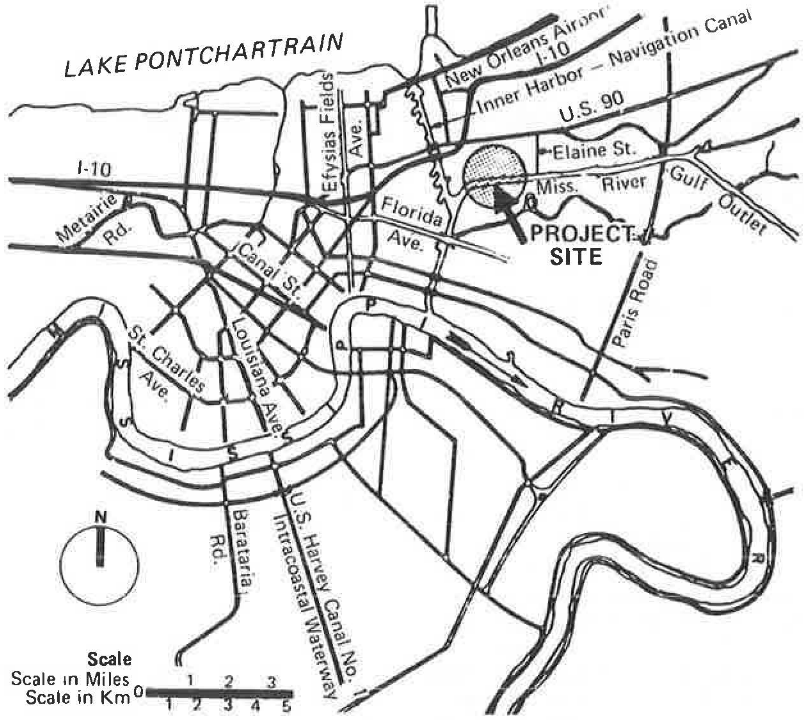

FIGURE 1 Project location map.

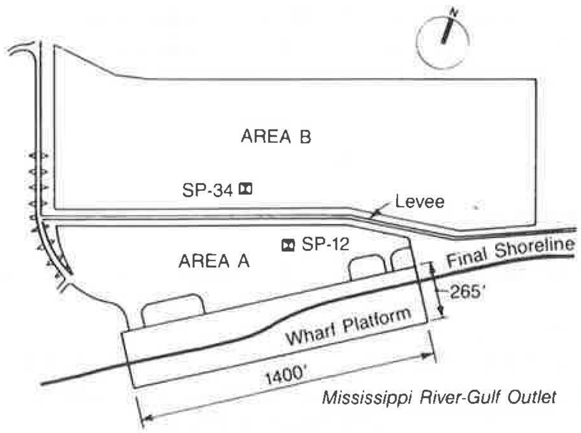

FIGURE 2 Site plan for Berths 4 and 5.

The most recent deposits consist of paludal, backswamp deposits. These deposits are composed of fine grained organic soils formed by sediments laid down in shallow areas by river floodwaters. Backswamp soils are generally slightly overconsolidated due to desiccation caused by alternate wetting and drying (*3*).

Subsoil Conditions

Figure 3 presents a typical soil profile for the project site. As shown in the figure, the subsoils include a surface stratum of very soft organic clays to approximately El. −14 (Mean Sea Level Datum). This is underlain by a very soft clay to silty clay containing numerous silt and sand lenses between approximately El. −14 and −41. Following are strata of loose clayey fine sand to soft sandy clay between El. −41 and −58. Below El. −58 is a stratum of medium dense to very dense silty fine sand.

Based on visual inspection and the results of laboratory testing, it is believed that these strata correspond to the following geologic deposits: backswamp deposits above El. −14; fluvial-marine deposits between El. −14 and −41; marine baysound deposits between El. −41 and −58, and Pleistocene deposits below El. −58.

FIELD AND LABORATORY INVESTIGATIONS

An extensive subsurface investigation program was undertaken at the Jourdan Road Terminal site in order to determine the depth of the soft clays and to obtain undisturbed samples for laboratory testing. Standard 3-in. diameter undisturbed samples and a small number of 5-in. diameter undisturbed samples were obtained. In addition, field vane shear tests were performed in selected locations within the soft clay stratum between El. −20 and −41 using a Nilcon Model 70 vane borer (*5*).

The laboratory testing program consisted of a large number of standard classification tests, unconfined compression (UC) tests, and unconsolidated undrained (UU) triaxial tests. Also included in the laboratory testing program were one-dimensional consolidation (odometer) tests, and several laboratory permeability tests on specimens cut vertically and horizontally from 5-in. diameter samples for comparing vertical and horizontal permeabilities.

SOIL PROPERTIES

Typical Atterberg Limits and natural water contents of the clayey soils are plotted versus elevation in Figure 3. Based on Atterberg Limit determinations, soft clays at the site are classified as CH and CL according to the Unified Soil Classification System (*6*), and have a pasticity index approximated by the equation $PI = 0.86\ (LL = 15)$. Organic contents for the

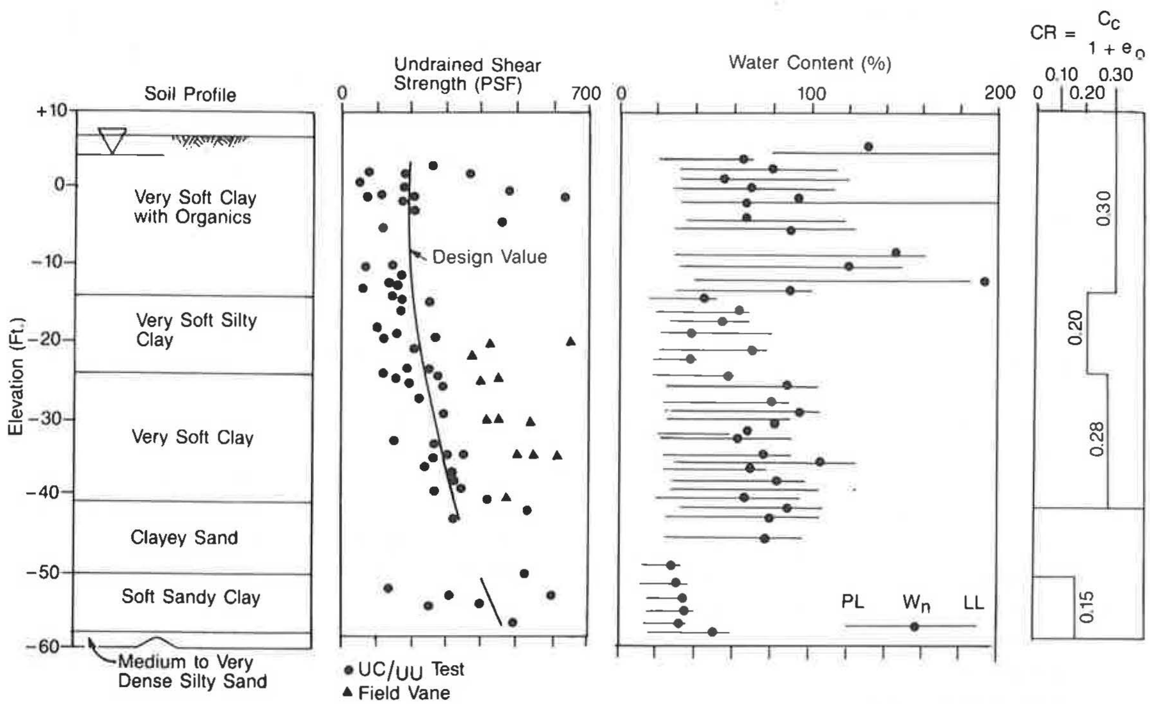

FIGURE 3 Generalized soil profile and laboratory test data.

organic clay stratum above El. −14 were generally less than 10 percent as determined by ignition loss.

The undrained shear strengths obtained from field vane tests and laboratory shear strength tests are also plotted in Figure 3. The field strengths, which were adjusted using Bjerrum's correction factor (*7*), are about 1.5 times the unconfined compression strengths obtained from laboratory tests on 5-in. undisturbed samples. This value is comparable to values presented by Arman et al. (*5*) for similar soils in New Orleans. Using an upper bound of the laboratory shear strength values, a $c/\overline{p}$ ratio of 0.20 was used to define the design shear strength profile for the normally consolidated clays below El. −14. For the soils above El. −14, which exhibited slight overconsolidation, an undrained shear strength value, *c*, of 200 psf was used in design.

The compression ratio, $CR = C_c/(1 + e_o)$, was found to vary linearly with the natural water content by the equation $CR = 0.0031\ W_n + 0.042$. The design values developed from this relationship are presented in Figure 3.

The coefficient of consolidation, c_v, obtained from the laboratory tests varied widely between 2 ft^2 per sec and 0.006 ft^2 per sec, reflecting the influence of inclusions of silts, sands, shell fragments and organic matter in the test specimens. A value of 0.03 ft^2 per day appeared to represent the average of a large number of data points and was chosen for design. However, for vertical drain installations the coefficient of horizontal consolidation, c_h, governs the rate of settlement. Due to the presence of silt and sand lenses in the clays it was expected that c_h would be larger than c_v. Based on the results of horizontal and vertical laboratory permeability tests, the ratio of c_h/c_v was estimated to range between 8 and 17. However, this high permeability ratio may not be representative of the actual field conditions because silt or sand lenses which extend the full length of the test specimen may not be continuous over the distance between vertical drains. Also, smearing of the soil immediately around the drain during drain installation would further reduce the field value of horizontal permeability. A ratio of $c_h/c_v = 2$ was selected for design since this value was considered more realistic than the laboratory test results, and was also considered reasonably conservative. Using the laboratory c_v value of 0.03 ft^2 per day, the design c_h value was thereby estimated to be 0.06 ft^2 per day.

CRITERIA FOR PRELOADING

It was anticipated that several feet of settlement, occurring over a period of more than 15 years, would result from the weight of the required embankment fill. Such settlements would lead to frequent and costly maintenance of pavements, railroad track bed, and utilities, and might also seriously affect port operations. To avoid, or at least minimize, these problems, a construction preload program was developed for Area A using vertical drains to complete preloading within a scheduled 9-mo construction preload period.

The design criteria required that the subsoils be preconsolidated to a total load of approximately 900 psf, which included a fill load of approximately 400 psf and a design live load of 500 psf. The design live load was treated as a long-term loading since it represented the load from shipping containers stacked for possibly prolonged periods of time over sizeable areas. In order to accelerate the consolidation of the subsoils, an additional load, or surcharge, of approximately 200 psf was applied. The criteria for full consolidation of the subsoils to the design load condition required approximately 80 percent consolidation under the surcharge loading.

DESIGN OF VERTICAL DRAINS

In the contract documents, wick drains were added as a construction alternative to the then conventional sand drains because of

- Their ease of installation,
- The environmental advantages resulting from the elimination of water jetting, and
- The competitiveness of their cost with sand drains.

Two types of wick drains, Alidrain™ and Geodrain™, were selected and incorporated in the design primarily because of their availability in the United States at that time. All drains were required to extend from a sand working mat at the ground surface to the dense sand stratum below El. –58 resulting in drains with a length of approximately 65 ft.

The Alidrain in use at the time of the Jourdan Road project was 3.9 in. wide by 0.28 in. thick and composed of a dual layered inner plastic core, perforated and studded to facilitate seepage through the drain and covered with a geotextile filter fabric. The Geodrain then in use was 3.9 in. wide by 0.16 in. thick, composed of a plastic core with longitudinal grooves on each side, and wrapped with filter paper. It should be noted that both the Alidrain and Geodrain have been modified by their manufacturers since the completion of the Jourdan Road project, and that these modifications may have an influence on drain performance.

The wick drain alternative was designed using procedures described by Moran et al. (*8*), based on numerical solutions developed by Barron (*9*) for consolidation by lateral flow to vertical drains. In this analysis the zone of influence of each drain is converted into a cylindrical volume having an equivalent diameter, d_e. For any time, t, after load application, a time factor, T_h, can be computed by the following equation:

$$T_h = \frac{c_h t}{(d_e)^2} \tag{1}$$

where c_h is the coefficient of consolidation for horizontal drainage.

For an equivalent drain diameter, d_w, and drain influence zone, d_e, the average percent consolidation, U, can then be determined by numerical solutions or by using graphical correlations between U and T_h (*9*).

The equivalent drain diameter for a wick drain can be determined as proposed by Hansbo (*10*), by the following equation:

$$d_w = \frac{2\ (drain\ width + drain\ thickness)}{\pi} \tag{2}$$

Based on this relationship, the equivalent diameter was determined to be approximately 2.8 in. for both the Alidrain and Geodrain. Using this diameter, the design required the drains to be installed in a triangular pattern at 5-ft spacing.

PRELOADING WITHOUT DRAINS

Preloading was also designated for Area B to prepare this site for future development. However, in this area the embankment preload would be left in place for many years before development of the site, providing sufficient time to preconsolidate the subsoils without the use of vertical drains.

At Area B the average preload, including fill and surcharge, was approximately 850 psf. The rate of settlement was determined using the classical Terzaghi one-dimensional consolidation theory, assuming relief of excess pore water pressures by vertical seepage through the clayey soils. A thin humus layer at about El. –6 with the organic clay stratum, and sand strata at about El. –42 and below El. –58 were assumed to provide effective lateral drainage. For these conditions, it was estimated that approximately 16 years would be required for 90 percent consolidation of the subsoils.

CONSTRUCTION

Construction of the Jourdan Road facility commenced in June 1980 with the award of an earthwork contract which included installation of drains and placement of fill and surcharge within Areas A and B. The contractor for the earthwork contract elected to use Alidrains for Area A.

Before installation of the wick drains, however, it was necessary to place a working mat over the soft organic soils for support of equipment loads. The working mat consisted of a reinforcing sheet of Mirafi 500X geotextile fabric beneath a 2-ft-thick layer of fill. Since the working mat also served as a drainage blanket, a clean, coarse-to-medium-size sand was used for this layer.

The installation of the drains followed the placement of the working mat, and was performed from June through September 1980. Placement of the remaining fill and surcharge at Area A followed the installation of the drains, and was performed from September 1980 to January 1981. The fill and surcharge was composed of a uniformly graded fine sand, known locally as "river sand." The surcharge was maintained for a period of approximately 12 months before removal and subsequent construction in this area.

Placement of fill and surcharge in Area B commenced in June 1980 and was completed in January 1981. At this area the surcharge was left in place after the completion of all construction for the port facility.

INSTRUMENTATION

Instrumentation installed to monitor performance of the preload program included 4 inductance-type subsurface settlement indicators and 4 sets of pneumatic-type piezometers within Area A, and a total of more than 50 settlement platforms at both Areas A and B.

The subsurface settlement indicators were composed of a 2-in.-diameter rigid plastic pipe within a 3-in. (inside diameter) corrugated polyethylene tubing to which wire inductance rings were attached at approximately 5-ft intervals. The subsurface settlement indicators extended to the dense sand stratum below El. –58 where a bottom inductance ring served as a reference for determining settlements of all other rings. Settlements were monitored from within the rigid plastic pipe using a Sondex Settlement Probe, Model Number 50819, from Slope Indicator Company.

Each group of piezometers included three piezometer sensors installed at different elevations within separate boreholes.

The piezometers used were Petur Instrument Company Model P-106 Canvaspack piezometers, which were furnished with a sand filter and bentonite pellet seal within a prepackaged fabric mesh.

Settlement platforms consisted of 3-ft × 3-ft by 1/8-in.-thick steel plates with a riser pipe protected by an outer steel casing. The settlement platforms were installed on the existing ground surface before placement of any fill. Subsurface settlement indicators and piezometers in Area A were installed after placement of the 2-ft-thick sand working mat.

DATA EVALUATION

Surface Settlement

Figure 4 presents plots of surface settlement versus time obtained from two typical settlement platform installations, including one settlement platform (SP-12) located in Area A, and a second (SP-34) located in Area B. The approximate locations of these settlement platforms are shown in Figure 2. Included in Figure 4 are plots of predicted settlements developed using the parameters and method of analysis described previously.

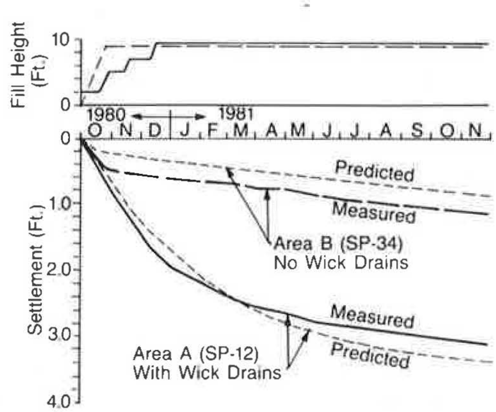

FIGURE 4 Surface settlement versus time from settlement platforms.

The height of fill and surcharge at both SP-12 and SP-34 was approximately 9.0 ft, corresponding to an applied load of about 1,080 psf and a total primary settlement estimated to be about 3.7 ft. The equal loading at both settlement platform locations allows a direct comparison of the settlement data obtained from Areas A and B.

As typified by the plots in Figure 4, total settlements measured in the wick drain area were generally less than predicted. After 9 mo of preloading, the data from SP-12 indicate a total settlement of 3.00 ft, and an average consolidation of the subsoils of approximately 80 percent. For this same period, the average consolidation determined from a large number of settlement platforms in the wick drain area was approximately 75 percent, somewhat less than the design criteria of 80 percent. To obtain additional consolidation, the preload period was extended from 9 to 12 mo by taking advantage of available slack in the construction schedule. At 12 mo the average consolidation at SP-12 had increased to approximately 85 percent, and for all settlement platforms was approximately 80 percent.

Within the same 12-mo preload period the total settlement measured at SP-34 in Area B was 1.18 ft, corresponding to an average consolidation of only 31 percent. The average consolidation from a large number of settlement platforms was 28 percent. Based on these data, an estimated 9 years would be required to achieve 80 percent consolidation in Area B. For 90 percent consolidation, approximately 14 years would be required.

Subsurface Settlement

Figure 5 presents subsurface settlement profiles obtained from Sondex installation "C" located within Area A, near settlement platform SP-12. Shown in the figure are settlement profiles at various times after September 24, 1980, the date of the initial instrument reading. Placement of fill and surcharge was completed at this location in mid-December 1980, as represented by the profile for Day 75. The profile for Day 418 was obtained after more than 11 mo of full surcharge loading.

The total height of fill and surcharge at Sondex "C" was approximately 9.0 ft, the same as at settlement platform SP-12. Thus, the settlement profiles from Sondex "C" can be directly correlated with the surface settlements measured at SP-12.

The settlement profiles indicate that approximately half of the observed settlements resulted from consolidation of the soft organic clay layer above approximately El. −14. Significant consolidation also occurred in the soft clayey soils between El. −14 and −41. Only 0.15 ft, or 5 percent of the total surface settlement, resulted from consolidation of the clayey sand and sandy clay strata below El. −41.

The compression of each stratum was determined by taking settlement differences between Sondex inductance rings located near the top and bottom of the stratum. The compression of the various clayey soil strata is plotted versus log time in Figure 6. These data show that compression of the lowermost stratum of clayey sand and sandy clay was completed, but that the remaining strata continued to consolidate. The linear relationship in the plots indicates that near the end of the preload period the three upper strata are still undergoing primary consolidation. Secondary compression, which should be evidenced by a break in the slope of the plots, apparently has not been reached within any of the three upper clay strata.

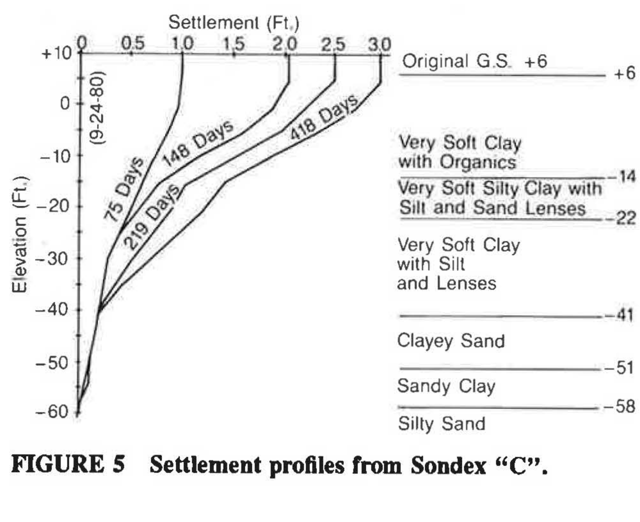

FIGURE 5 Settlement profiles from Sondex "C".

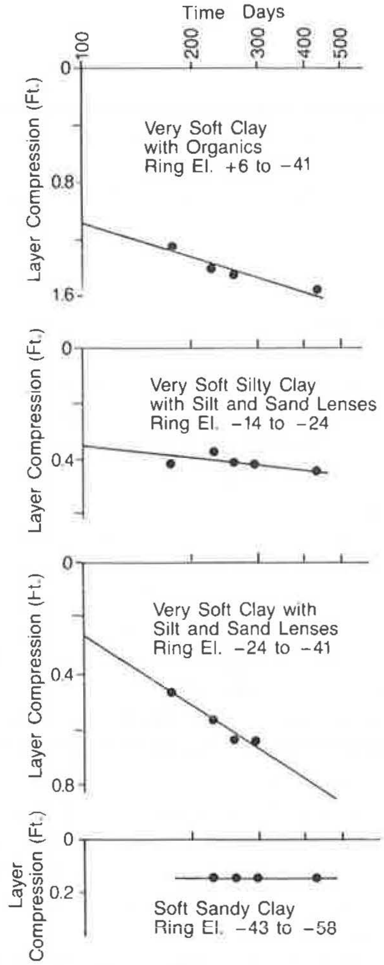

FIGURE 6 Layer compression versus log time from Sondex "C."

The layer compression data presented in Figure 6 were compared with calculated values of total layer settlements to estimate the percent consolidation for each of these strata. The results of this analysis are summarized in Table 1. After more than 11 mo of preloading, the clayey soils above El. −41 had reached an average of about 79 percent consolidation. Compression of the lowermost stratum was approximately equal to the calculated settlement, substantiating the conclusion drawn from Figure 6, that this stratum was fully consolidated.

Piezometer Data

Piezometer group "C" was located in Area A near SP-12 and Sondex "C" and included three piezometer sensors, installed at El. −12, −23, and −35. Figure 7 presents plots of the pore water pressures measured by the piezometers during the preload period. The pore water pressure changes during and after placement of the fill and surcharge are similar for all three piezometer sensors. All show a relatively rapid initial rate of pore water pressure dissipation followed by a gradually declining rate during the preload period. At the end of almost 12 months of preloading, all showed some excess pore water pressure, indicating that the subsoils were not fully consolidated.

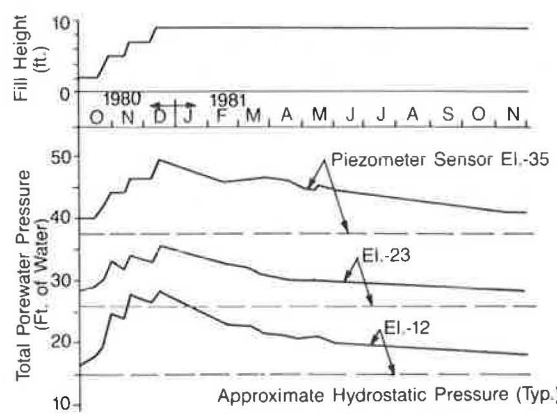

FIGURE 7 Pore water pressure versus time for piezometer Group "C."

Figure 8 presents plots of excess pore water pressures plotted at the depth of piezometer sensors using the data from Figure 7. Shown are excess pore water pressures determined at 0, 3, 6, and 12 mo after surcharge placement was completed. Initially, the pore water pressure dissipation was significantly faster at the sensor located at El. −22, but after 12 mo all piezometers showed similar values. It is believed that this variability resulted from the position of the piezometer sensor with respect to the surrounding drains. Although efforts were made to install the piezometers at the centroid of three drains, the location at depth could have shifted due to inclination of the drains or the piezometer drill hole.

After 12 mo of preloading, the three sensors in Piezometer Group "C" indicted an average consolidation of approximately 80 percent within the depths monitored.

TABLE 1 LAYER CONSOLIDATION ESTIMATED FROM SONDEX "C"

Soil Stratum	Inductance Ring Elevation (ft)	Total Estimated Settlement (ft)	Measured Settlement 11/17/81 (ft)	Average Consolidation (%)
Organic clay	+6 to −14	2.04	1.56	77
Silty clay with silt and sand lenses	−14 to −24	0.59	0.45	76
Clay with silt and sand lenses	−24 to −43	0.95	0.81	85
Clayey sand and sandy clay	−43 to −58	0.14	0.15	100
Total		3.72	2.97	79

Coefficient of Consolidation

After placement of the fill and surcharge, the relatively uniform rate of surface settlement of approximately 0.04 ft per month measured in SP-34 (Figure 4) closely matched the predicted rate, and provided evidence supporting the c_v value and drainage boundary conditions assumed in design of the preload program for Area B.

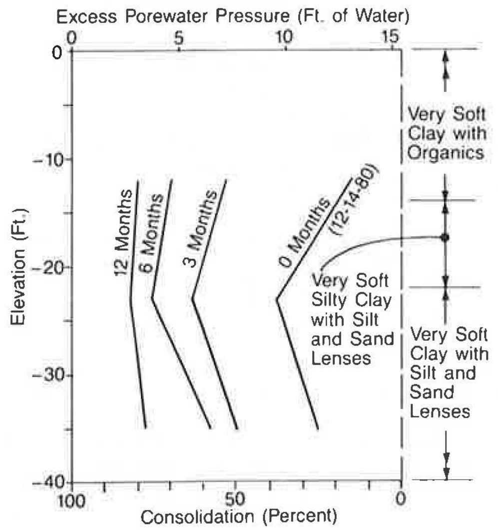

FIGURE 8 Excess pore water pressure versus depth for piezometer Group "C."

For Area A with wick drains, however, the measured rate of settlement after placement of fill and subcharge was less than predicted. The settlement platform, Sondex subsurface settlement indicator, and piezometer data all indicated that about 12 mo of preloading was required to obtain an average 80 percent consolidation of the subsoils, rather than 9 mo as estimated during design. These data suggest that the field c_h value is less than that assumed in the design of the wick drain installation.

The Sondex and piezometer data further showed that consolidation due to lateral flow to the drains occurred at similar rates in the various clayey soil strata above El. −41. Based on this observation it is concluded that the c_h value was approximately the same for all of these strata. This average c_h value can be estimated from the field data using Equation 1, modified as follows:

$$c_h = \frac{T_h d_e^2}{t} \tag{3}$$

where

d_e = 1.05 (5 ft) = 5.25 ft,
t = 12 mo = 365 days, and
T_h = 0.51 for U = 80 percent and n = 24.

From this equation the field c_h value was computed to be 0.039 ft^2 per day, which was somewhat less than the design value of $c_h = 0.060$ ft^2 per day. Using the computed c_h value, the ratio between c_h and the laboratory c_v value is then

$$\frac{c_h}{c_v} = \frac{0.039}{0.030} = 1.3$$

Also, using the field c_h value computed above, the coefficient of horizontal permeability, k_h, can be computed using the following relationship:

$$k_h = \frac{c_h a_v \gamma w}{1 + e} \tag{4}$$

where

a_v = coefficient of compressibility obtained from laboratory consolidation tests,
γw = unit weight of water, and
e = void ratio.

From Equation 4, k_h was estimated to be 4×10^{-8} in./s for the organic clays above El. −14, and 2×10^{-8} in./s for the clays with silt and sand lenses between El. −14 and −41.

Factors influencing lateral flow which may have contributed to the low c_h and k_h values computed for the Jourdan Road facility include

- Smear of soil around the drains,
- Discontinuity of silt and sand lenses within the clay strata, and
- Conductivity of the vertical drains and surface drainage layer.

The effects of these various factors require further study. However, the Jourdan Road results suggest that conservative c_h/c_v ratios should be used in design of wick drain installations unless more reliable data for the field c_h value are available. The ratio of c_h to c_v developed from laboratory permeability and consolidation tests on clayey soils from the New Orleans site were found to be unreliable for wick drain applications.

SUMMARY AND CONCLUSIONS

The construction of the Jourdan Road Terminal in New Orleans required the preloading of a 10-acre container storage area (Area A) using vertical drains to accelerate consolidation of approximately 60 ft of soft clayey soils. A total of approximately 2 million linear feet of prefabricated wick drains were installed within this area. A proposed future storage area (Area B) of approximately 32 acres was preloaded without the use of vertical drains.

Instrumentation data obtained during construction clearly illustrated the effectiveness of wick drains for accelerating consolidation of the soft subsoils. After a 12-mo preload period, Area A with wick drains had settled approximately 3.0 ft, with an average consolidation of the subsoils of about 80 percent under the surcharge load. In comparison, Area B without wick drains settled approximately 1.2 ft, and had an average consolidation of about 28 percent. In Area B approximately 9 years would be required to achieve 80 percent consolidation.

Consolidation occurred rapidly in the clayey sands and sandy clays below El. −41. However, the rate of consolidation of the soft clayey soils above El. −41 was somewhat less than predicted, requiring an extension of the preload period from 9 to 12 mo.

Data from the Sondex subsurface settlement indicators and piezometers indicated that consolidation occurred at similar rates in the various clayey soil strata above El. −41. The c_h value for these strata, calculated from the field data, was estimated to be 0.039 ft^2 per day, and corresponded to a c_h/c_v ratio of 1.3. This c_h/c_v ratio was lower than the value of 2.0 used in design, and considerably lower than the ratio determined from laboratory permeability and consolidation tests.

The coefficient of horizontal permeability, calculated from the field data, was estimated to be 4×10^{-8} in./s for the organic clays above El. −14, and 2×10^{-8} in./s for the clays with silt and sand lenses between El. −14 and −41.

After 12 mo of preloading, the subsoils were consolidated sufficiently to meet design criteria and to permit removal of the surcharge and continuation of construction. The Jourdan Road Terminal facility has now been in successful operation for more than 5 years.

ACKNOWLEDGMENTS

The Jourdan Road Terminal facility is owned and operated by the Board of Commissioners of the Port of New Orleans. Parsons, Brinckerhoff, Quade & Douglas, Inc. was the design engineer for Berths 4 and 5, and conducted the instrumentation monitoring program. Atlas Construction Company, Inc. was prime contractor for the Earthwork Contract, with Vibroflotation Foundation Company as subcontractor for installation of the wick drains. Eustis Engineering Company of Metairie, Louisiana, performed the borings and laboratory soils testing.

REFERENCES

1. R. J. Castelli, S. K. Sarkar, and G. A. Munfakh. Ground Treatment in the Design and Construction of a Wharf Structure. *Proc., Symposium on Advances in Piling and Ground Treatment for Foundations,* Institution of Civil Engineers, London, England, March 1983, pp. 275–281.
2. S. K. Sarkar, G. A. Munfakh, and R. J. Castelli. Building a Port Under Unusual Site Conditions. *Proc., Ports 83 Specialty Conference,* ASCE, New Orleans, La., March 1983, pp. 269–283.
3. C. R. Kolb and W. G. Shockley. Engineering Geology of the Mississippi Valley. *Transaction,* ASCE, Vol. 124, 1959, pp. 633–645.
4. R. L. Montgomery. *Correlation of Engineering Properties of Cohesive Soils Bordering the Mississippi River from Donaldsonville to Head of Passes, Louisiana.* AD A035019. Army Engineer Waterways Experiment Station, Vicksburg, Miss., June 1974.
5. A. Arman, J. Poplin, and N. Ahmad. A Study of the Vane Shear. *Proc., ASCE Specialty Conference on In-Situ Measurement of Soil Properties,* Vol. I, June 1975, pp. 93–120.
6. A. Casagrande. Classification and Identification of Soils. *Transaction,* ASCE, 1948, pp. 901–992.
7. L. Bjerrum. Embankments on Soft Ground. *Proc., ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures,* Vol. II, June 1972, pp. 1–54.
8. Moran, Proctor, Muesser, & Rutledge. *Study of Deep Soil Stabilization by Vertical Sand Drains.* Department of the Navy, Bureau of Yards and Docks, June 1958.
9. R. A. Barron. Consolidation of Fine-Grained Soils by Drain Wells. *Transaction,* ASCE, Vol. 113, 1948, pp. 718–754.
10. S. Hansbo. Consolidation of Clays by Band-Shaped Prefabricated Drains. *Ground Engineering,* Vol. 12, No. 5, 1979, pp. 16–23.

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Performance of a Prefabricated Vertical Drain Installation Beneath an Embankment

Z. KYFOR, J. MASI, AND R. GEMME

An embankment was constructed over foundation soils consisting of marls overlying soft clayey silts. Analyses of the foundation conditions indicated that the embankment could not be constructed safely within the required time frame of 9 mo without foundation treatment. In addition, the embankment would be subjected to a large amount of settlement. It was decided to use prefabricated vertical drains spaced at 4.0, 5.5, and 7.5 ft on centers in order to accelerate consolidation of the soft foundation soils. This would accomplish a twofold purpose of transitioning differential settlement behind a pile-supported abutment and developing the necessary foundation strength required to support the embankment. The embankment was constructed in two stages with waiting periods. Performance of the drains was monitored with piezometer and settlement recording devices. This instrumentation indicated that the prefabricated vertical drains performed as expected and are a viable alternative to conventional vertical sand drains. This paper presents an evaluation of the performance and effectiveness of the prefabricated vertical drain system.

When highway embankments are constructed over weak foundation soils, stability and settlement problems can be expected. If time is of no importance, the embankments can be constructed slowly without special foundation treatment. However, in most cases required scheduling can add considerable cost to a project. Foundation treatments implemented to reduce construction time have proven to be cost effective.

Vertical drains are commonly used to accelerate consolidation of fine-grained foundation soils. The time to reach a given degree of consolidation is related to the permeability of the foundation soils and the square of the length of the maximum drainage path. As the length of the drainage path increases, the time required to reach a given degree of consolidation increases. Although soil permeabilities cannot be changed, the drainage path can be shortened by the use of vertical drains.

This paper examines the treatment design and performance of an embankment foundation that was part of a major highway project that entailed the widening, relocation, and reconstruction of an existing interchange. The project is located in the city of Syracuse at the northeastern end of Onondaga Lake in New York State (Figure 1). The embankment is the north approach to a 2,000-ft-long pile-supported structure. The maximum height of the embankment at the bridge abutment is 35 ft.

A maximum 9-mo time period was imposed on construction of the approach embankment in order to complete this project within the scheduling constraints. Analyses of the embankment loading indicated that settlement and stability would be a problem. To meet the 9-mo construction time schedule, a prefabricated vertical drain system was used in conjunction with stage construction.

New York State Department of Transportation, Soil Mechanics Bureau, Building 7, State Campus, 1220 Washington Avenue, Albany, N.Y. 12232.

BACKGROUND

Geology of the Syracuse Vicinity

During the last glacial era, the Syracuse area was covered by a glacial lake. This glacial lake basin began filling with lacustrine silts and clays by underwater sedimentation which has never been subjected to more than its own weight. As the glacier began receding some 10,000 years ago, some of the low-lying areas were occupied by smaller lakes, the present Onondaga Lake being one of them. During the glacial retreat, the surrounding areas of Onondaga Lake began filling with marl, which is a mixture of calcium carbonate, shells, silt, and clay. After the marl was laid down, the formation of peat began. The swampy environment, which exists to this day, favored the preservation of the organic material. In addition to the natural causes of deposition the foundation conditions are further complicated by man-made deposits. Landfill operations in the Syracuse area have dumped garbage and miscellaneous fill in varying quantities throughout many parts of the area.

General Foundation Conditions

Ten drill holes and one undisturbed sample drill hole were dug in this area. This subsurface investigation generally revealed a surface layer of miscellaneous fill (brick, cinders, sand, etc.) ranging in thickness from 5 to 10 ft overlying 25 to 30 ft of marl. This marl is underlain by 30 ft of soft clayey silt over compact sands and silts extending to bedrock. The embankment profile and foundation stratigraphy are shown in Figure 2. Moisture contents obtained from the drill hole samples were plotted versus elevations and are shown in Figure 3.

ENGINEERING PROPERTIES OF FOUNDATION SOILS

Sampling

Hydraulically driven Shelby tube samples were obtained for the purpose of conducting tests to determine the engineering properties of the soft foundation soils. Laboratory testing

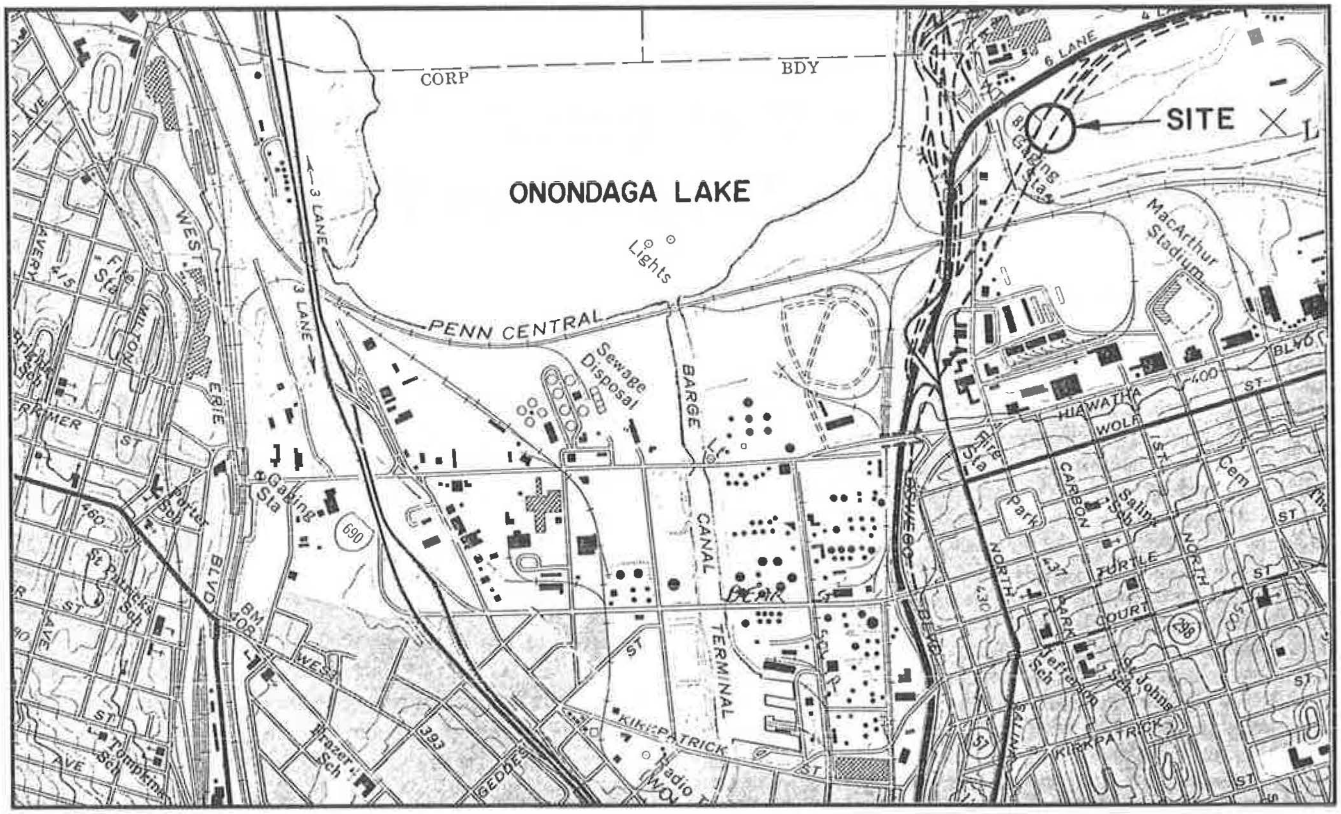

FIGURE 1 Project location plan—Syracuse, New York.

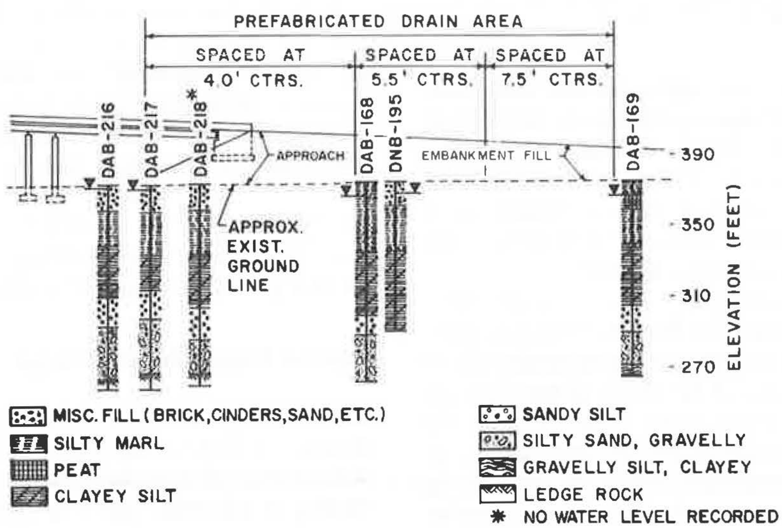

FIGURE 2 Soil profile.

consisted of classification tests, oedometer tests, and triaxial compression strength tests.

Considerable difficulty was encountered in trimming the Shelby tube samples for consolidation and strength testing. Many of the marls were so granular that it was difficult to test them. The marl samples that lent themselves to being tested were those of a fine-grained nature. Some of the clayey silt samples would start to flow upon extrusion from the Shelby tubes due to a very low clay content. The clayey silt samples that lent themselves to being tested were those with a higher percentage of clay. Hydrometer analysis performed on the clayey silt samples indicated that 15 to 20 percent passed the 0.002-mm size.

Classification Characteristics

Classification tests were performed to aid in the identification of the various strata. These tests consisted of Atterberg limits, specific gravity, wet density, and natural moisture content. A summary of the results is presented in Table 1. According to the Unified Soil Classification System the soft clayey silts would

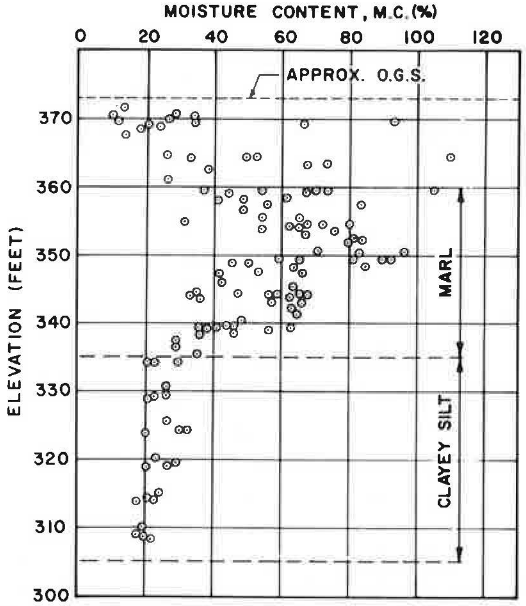

FIGURE 3 Elevation versus moisture content (prefabricated drain area).

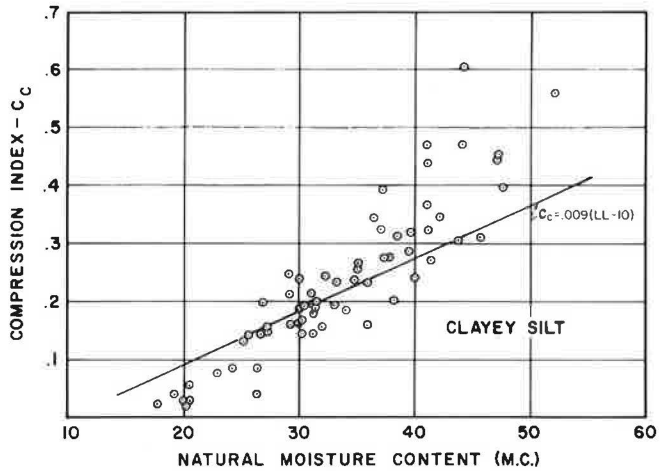

FIGURE 4 Compression index for clayey silt versus moisture content (total project area).

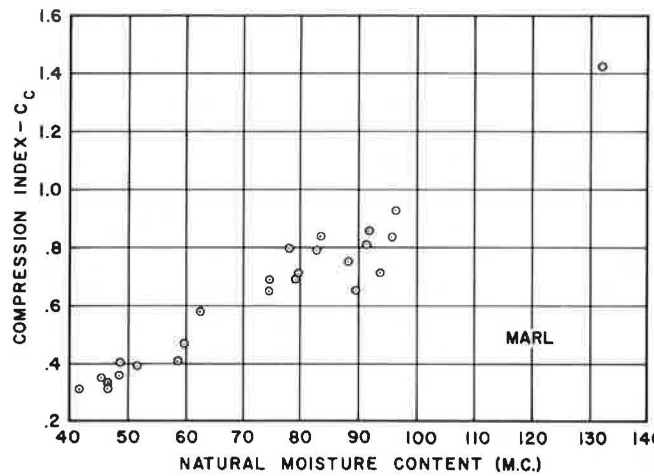

FIGURE 5 Compression index of marl versus moisture content (total project area).

be classified as CL-ML (inorganic clayey silts of slight to low plasticity).

Consolidation Characteristics

The consolidation characteristics of the foundation soils determined from oedometer tests indicated that large settlements could be expected under the embankment loads. The preconsolidation pressures estimated, using the Casagrande technique, showed that they were approximately equal to the effective vertical overburden pressures, confirming the geologic history of the deposit as being normally consolidated.

The compression indices, C_c's, for the clayey silts and marls were obtained from the oedometer tests and are plotted versus the natural moisture contents. These results are shown in Figures 4 and 5. Coefficients of vertical consolidation, c_v's, for the clayey silts were plotted versus the natural moisture contents and are shown in Figure 6. Consolidation tests on the marl indicated that the c_v would be greater than 1.0 ft^2 per day. The consolidation parameters presented in Figures 4, 5, and 6 represent the entire project area and not just the area under investigation (i.e., other undisturbed laboratory testing results from drill holes not in the immediate vicinity of the 35-ft-high embankment were also included in the summaries). This was done because the subsoils throughout the area are very similar.

Strength Characteristics

The Soil Mechanics Bureau's past experience with the marls indicated that they drain relatively quickly under loading and can be assumed to act drained, as a sandy soil. This assumption was based on embankment construction instrumentation monitoring results of an interchange in the same area in 1951. A friction angle of 28 degrees was assigned to the marl because of its loose granular nature and previous consolidated drained triaxial compression tests.

The undrained shear strengths for the soft clayey silts were determined by consolidated isotropic undrained (CIU) triaxial compression tests. Since this clayey silt deposit is normally

TABLE 1 SOIL CLASSIFICATION RESULTS (PREFABRICATED DRAIN AREA)

Soil Type	Liquid Limit (L.L.)	Plasticity Index (P.I.)	Wet Density (pcf)	Specific Gravity of Solids	Natural Moisture Content (%)
Marl	40–60	8–13	90–120	2.65±	50–100
Clayey silt	21–30	5–16	110–130	2.75±	20–35

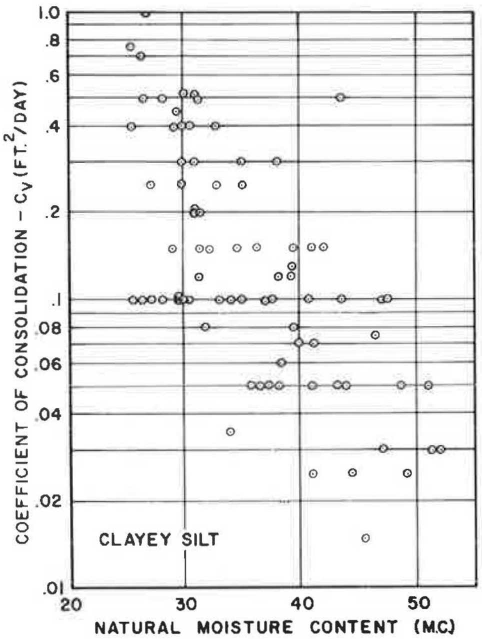

FIGURE 6 Coefficient of consolidation for clayey silt versus moisture content (total project area).

consolidated, the undrained shear strength could be expected to increase linearly with depth. This trend, defined by the c/p ratio and based on laboratory testing, was determined to be equal to 0.25. Previous testing in the area indicated that the clayey silt had sensitivity values of 2 to 3 (considered insensitive).

PREDICTED EMBANKMENT PERFORMANCE

Stability

Embankment stability analyses were performed in terms of total stress analysis ($\phi = 0$) using the Modified Bishop Method of Slices. Analyses indicated that without foundation treatment the proposed 35-ft-high approach embankment, having one vertical on two horizontal side slopes, could not be safely constructed within a 9-mo period because of insufficient strength gain in the foundation soils. Subsequent analyses indicated that the embankment fill could be safely constructed to a height of 20 ft with no rate restrictions.

Total Settlement

The total estimated settlement beneath the centerline of the maximum 35-ft-high embankment was in the range of 5 to 6 ft. Most of this settlement (60 to 70 percent) was predicted to be attributed to consolidation of the marl layer with the remainder occurring in the clayey silt layer.

Time Rate of Settlement

Previous experience in this area indicated that the marl layer would consolidate as fast as the fill was placed. As a basis for design the marl layer was assigned a value of $c_v = 1.0$ ft^2 per day and the clayey silt layer was assigned a value of $c_v = 0.12$ ft^2 per day based on results shown in Figure 6.

FOUNDATION TREATMENT

The construction schedule established a time constraint of 9 mo from start of embankment construction to start of paving operations requiring that the embankment be stable and without detrimental differential pavement settlement. It was decided that a vertical drainage system would provide the best method of accelerating the settlement rate to meet this schedule. The contract plans provided the option of installing 12-in.-diameter augered or 18-in.-diameter jetted sand drains. Initially the contractor selected the 12-in. augered sand drain, but later proposed the use of a prefabricated vertical drain system. At that time prefabricated drains had been used only on a few projects in the United States and none in New York State. A prefabricated drain known as Alidrain™ was proposed by the contractor. Alidrain is a proprietary item consisting of a band shaped plastic core wrapped in a geotextile.

Considering a cost savings of approximately $100,000 (Table 2) and a review of other documented experiences with the use of these drains, it was decided to accept the contractor's proposal. In addition, the contractor stated that he would take the responsibility for any increased construction time if delays occurred as a result of less effective performance of the Alidrain.

It was proposed to construct the embankment in two stages. The first stage consisted of a maximum 20-ft-high fill, the safe height determined from stability analyses. After achieving 90 percent dissipation of excess pore water pressures (equivalent

TABLE 2 SAND DRAIN VERSUS PERFABRICATED DRAIN COSTS

Description	Quantity	Unit Price ($)	Amount ($)
Contract items to be deleted			
12-in.-diameter vertical sand drain	116,500 ft.	4.70	547,550.00
Collector drains	12,108 ft.	4.00	48,432.00
Embankment-in-place (replaced by stone blanket)	3,250 yd^3	9.23	29,997.50
Total			625,979.50
Contract items to be added			
Prefabricated drains	245,076 ft.	1.69	414,178.44
Stone blanket—north abutment	2,178 yd^3	35.16	76,578.48
Stone blanket—south abutment	1,072 yd^3	31.17	33,414.24
Total			524,171.16

NOTE: Above costs include both north and south abutment approaches.

to strength gain) in the foundation soils, construction of the remainder of the fill (maximum 15 ft additional fill) could be completed.

PREFABRICATED DRAINS

Design

The problem of designing a vertical drain system is to determine the drain spacing that will give the required degree of consolidation in a specified time. Prefabricated vertical drain design was performed using the Barron-Kjellman formula (*1*):

$$t_h = \frac{D^2}{8c_h}\left[\frac{\ln D/d}{1-(d/D)^2} - \frac{3-(d/D)^2}{4}\right]\ln\frac{1}{1-U_r}$$

where

t_h = time required to achieve U_r,
D = diameter of the zone of influence of the drain,
c_h = coefficient of horizontal consolidation,
U_r = average degree of consolidation by radial drainage alone, and
d = equivalent diameter of prefabricated drain.

Equivalent Sand Drain Diameter (d)

The equivalent sand drain diameter for a prefabricated drain is that diameter that will produce the same time rate of consolidation as that of a sand drain of equal diameter. The Soil Mechanics Bureau conducted a laboratory testing program to determine the range of equivalent sand drain diameters for various perfabricated drains (*2*). The testing program consisted of performing large diameter consolidation tests (Figure 7)

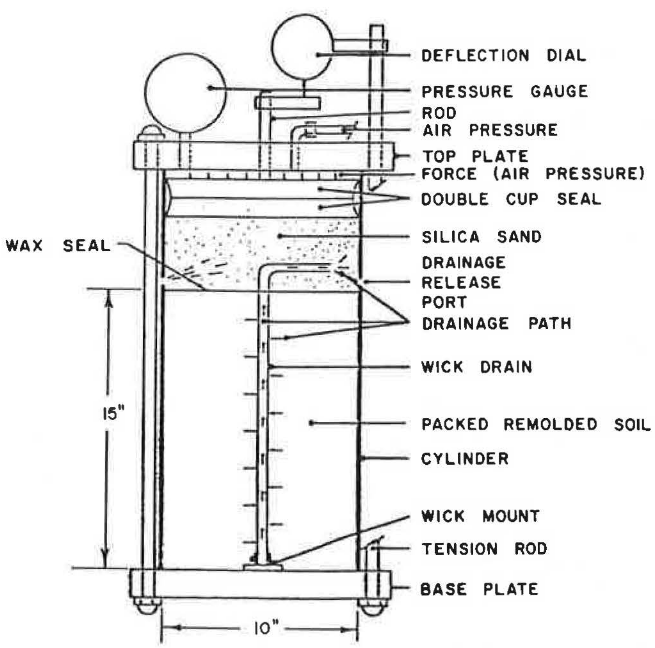

FIGURE 7 Wick drain consolidometer.

using three types of remolded soils. The results of this testing program for the Alidrain are given in Table 3. Based on these results, it was decided that the equivalent sand drain diameter (d) for the Alidrain would be 2 in.

Drain Layout

A square drain pattern was selected for this project because it was easier to lay out and control in the field. The effective diameter of the drain was taken as 1.13 S, where S is the drain spacing.

TABLE 3 RANGE OF EQUIVALENT SAND DRAIN DIAMETERS FOR AN ALIDRAIN

Soil Type	*MC* (%)	c_h (ft^2 per day)	Equivalent Sand Drain Diameter (in.)
Organic silty clay (westway soil)	50	0.03	2.50–1.37
Manufactured clay	40	0.04	2.27–1.05
Peat	338	0.04	1.89–1.56

Coefficient of Horizontal Consolidation (c_h)

An important parameter in designing a vertical drain system is the coefficient of horizontal consolidation (c_h). In practice, however, it is often difficult to obtain a realistic estimate of this key parameter. For design, the coefficient of horizontal consolidation c_h was taken as being equal to the coefficient of vertical consolidation (c_v), which is considered conservative. At a later date, block permeability tests (*3*) similar to the permeability test developed at MIT (*4*) were performed and the results are given in Table 4. The block permeability test setup is shown in Figure 8.

These tests indicated that a c_h equal to approximately three times the design c_v value would have been more appropriate. The predicted time rate of settlement for drains spaced on 4-ft centers for $c_h = c_v = 0.12$ ft^2 per day and for $c_h = 0.3$ ft^2 per day is shown in Figure 9.

Drain Spacing

The prefabricated drain spacing varied from 4 ft at the abutment to 7.5 ft, 274 ft away from the abutment (Figure 2). The closer spacing was used in the abutment area for stability purposes and the spacing was transitioned back to a larger spacing in order to minimize differential settlements because the abutment was pile supported.

INSTRUMENTATION PROGRAM

An instrumentation program was initiated to monitor the embankment foundation and evaluate the prefabricated drains. Four pneumatic piezometers, four pneumatic settlement systems (surface type), and one pneumatic settlement system (subsurface type) were installed. All piezometers were installed in the soft clayey silt layer. None were installed within the marl stratum because previous experiences indicated

TABLE 4 BLOCK PERMEABILITY TEST RESULTS

Depth (ft)	Soil Description	MC (%)	Specific Gravity	Limits LL	Limits PI	Percent Passing 0.002 mm	*Kh/Kv*
39.2	Gray brown clayey silt	27.0	2.75	19.3	6.1	20.5	3.14
40.1	Gray brown clayey silt	26.2	2.76	17.4	3.9	16.6	3.86

NOTE: Sample is extruded from Shelby tube and then trimmed to form a 2-in. cube.

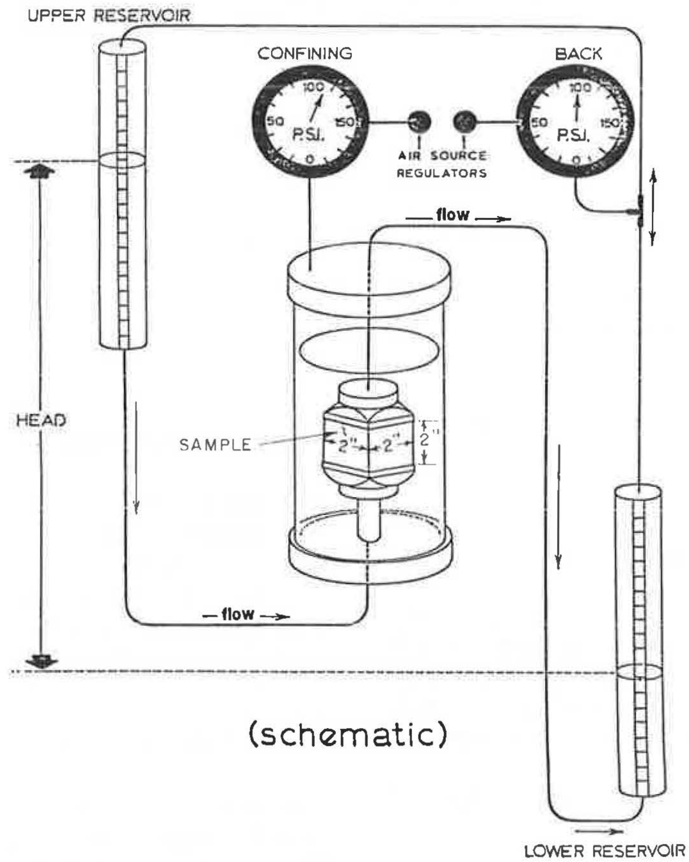

FIGURE 8 Block permeability test apparatus.

that excess pore pressure dissipation would occur as fast as the loads were applied.

INSTALLATION OF PREFABRICATED DRAINS AND INSTRUMENTATION

The ground surface was stripped of vegetation and a 2-ft granular drainage blanket was placed in the prefabricated drain area (*5*). Initial attempts to install drains failed due to obstructions encountered in the miscellaneous fill. Consequently, preaugering to a depth of 10 to 15 ft became necessary. Upon completion of the preaugering operation, it took approximately 1 min to install each Alidrain to a depth of approximately 72 ft to reach the bottom of the soft clayey silt layer. A total of 2,635 drains were installed.

After the drain installation was completed, piezometers and settlement systems were installed at the locations shown in Figure 10 and at the elevations shown in Figure 11. Actual locations were adjusted in the field to position piezometers at equal distances between drains to measure maximum pore pressures.

INSTRUMENTATION MONITORING

First Stage

Construction of the first stage started on June 21, 1982. The 20 ft of first-stage embankment fill was completed in approximately 30 days. There was a 4-mo waiting period during which the piezometers and settlement systems were monitored.

The amount of total settlement that occurred during the embankment construction and the subsequent 114-day (4± mo) waiting period is shown in Figure 9. The amount of settlement that had taken place in the 4.0-, 5.5-, and 7.5-ft drain spacing areas was 4.7 ft, 4.2 ft, and 3.2 ft, respectively. It was unfortunate that the subsurface settlement gauge (SS5) malfunctioned shortly after embankment construction began, because information from this instrument would have indicated

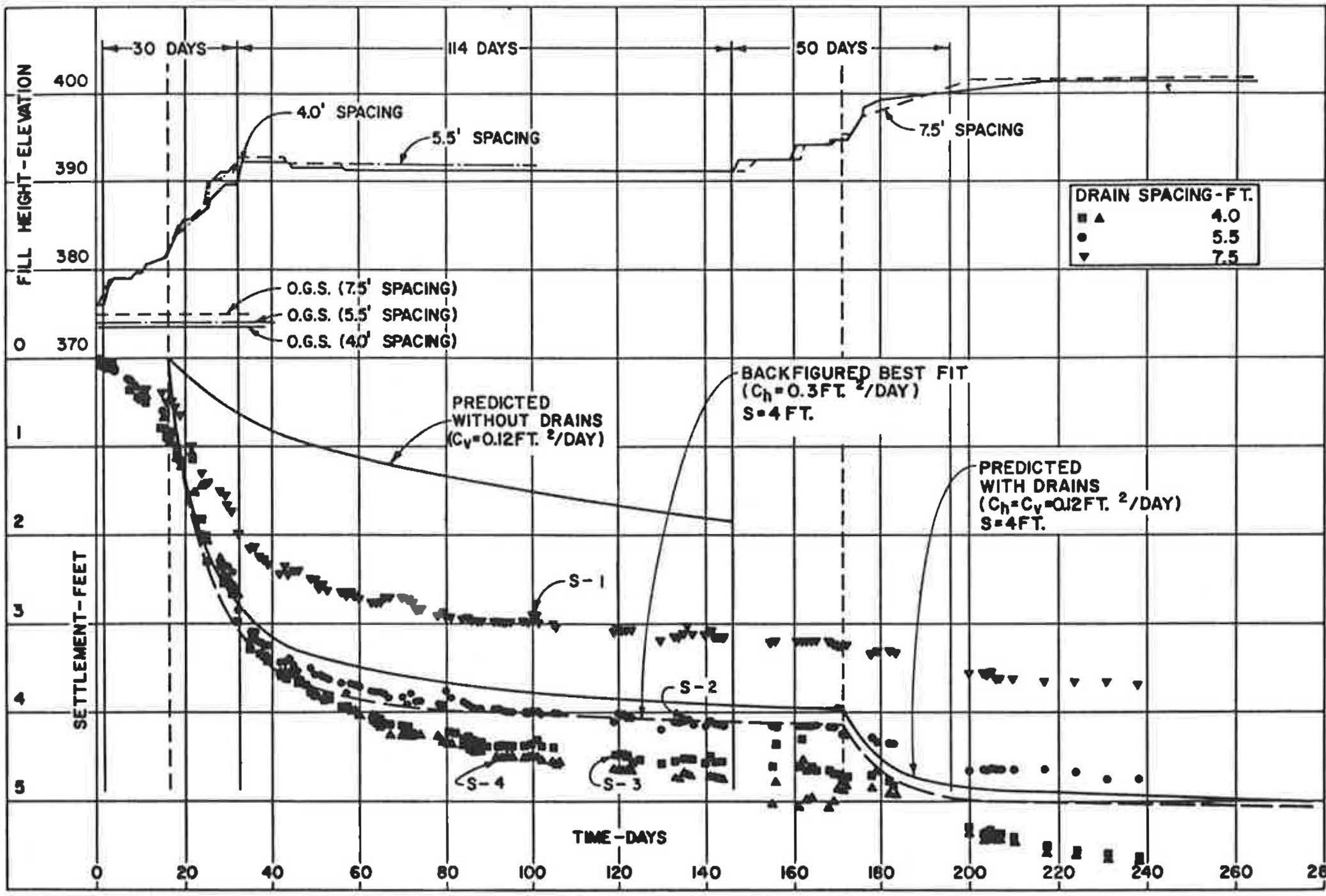

FIGURE 9 Predicted and measured settlements.

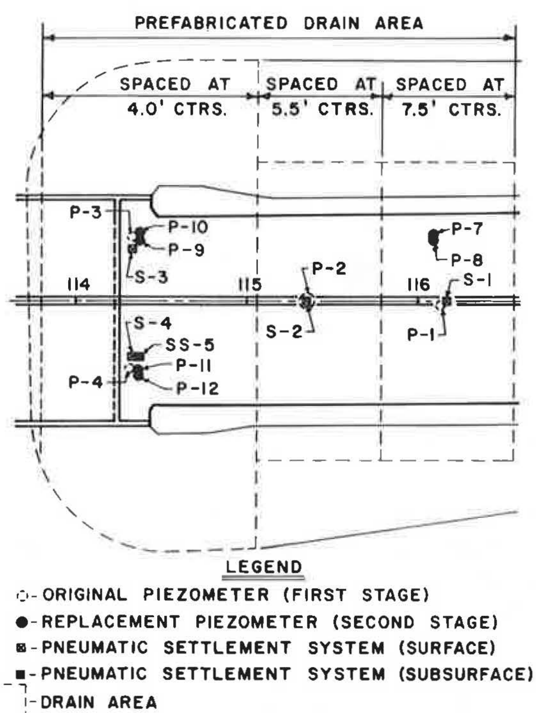

FIGURE 10 Location of prefabricated drains and instrumentation.

what percentage of the total settlement was occurring in the clayey silt. Without this information, any evaluation of the effectiveness of the prefabricated drains could not be made based on settlement data. Therefore, piezometer data were used for this purpose.

Figures 12, 13, and 14 show the measured excess pore pressure response to the fill placement. Piezometers P-1, P-2, P-3, and P-4 were functioning normally at the start of embankment construction. Piezometer P-4 ceased to operate within 20 days. The measured excess pore pressures increased in piezometers P-1, P-2, and P-3 as the embankment height increased, as would be expected.

At the end of the first stage embankment loading the measured excess pore pressures at P-1, P-2, and P-3 were 20 psi, 15 psi, and 9 psi, respectively. If dissipation of excess pore pressures had not taken place, the 20± ft of fill would have produced a maximum excess pore pressure of approximately 20 psi. The difference between the theoretical maximum and the measured excess pore pressure at the end of the loading period clearly indicates that dissipation was taking place as expected. Piezometer P-1, which was located in the 7.5-ft drain spacing area, showed virtually negligible dissipation of pore pressure at the end of the first stage fill, and P-3, which was located in the 4.0-ft spacing area, indicated that 50 percent of the expected excess pore pressure had dissipated at that time.

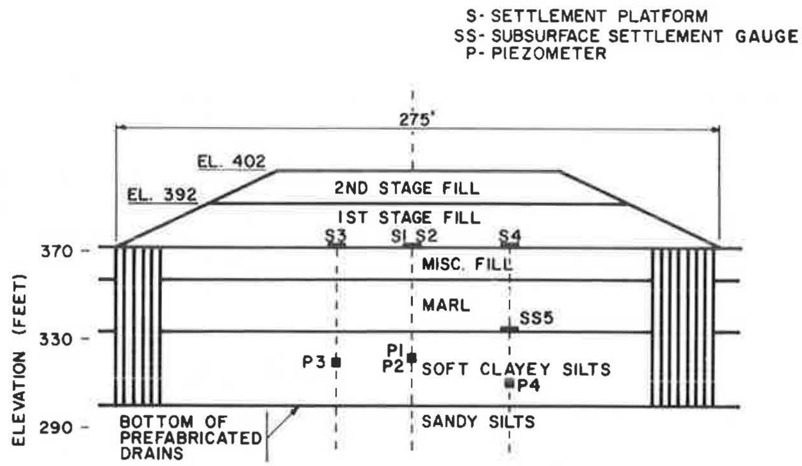

FIGURE 11 Embankment section.

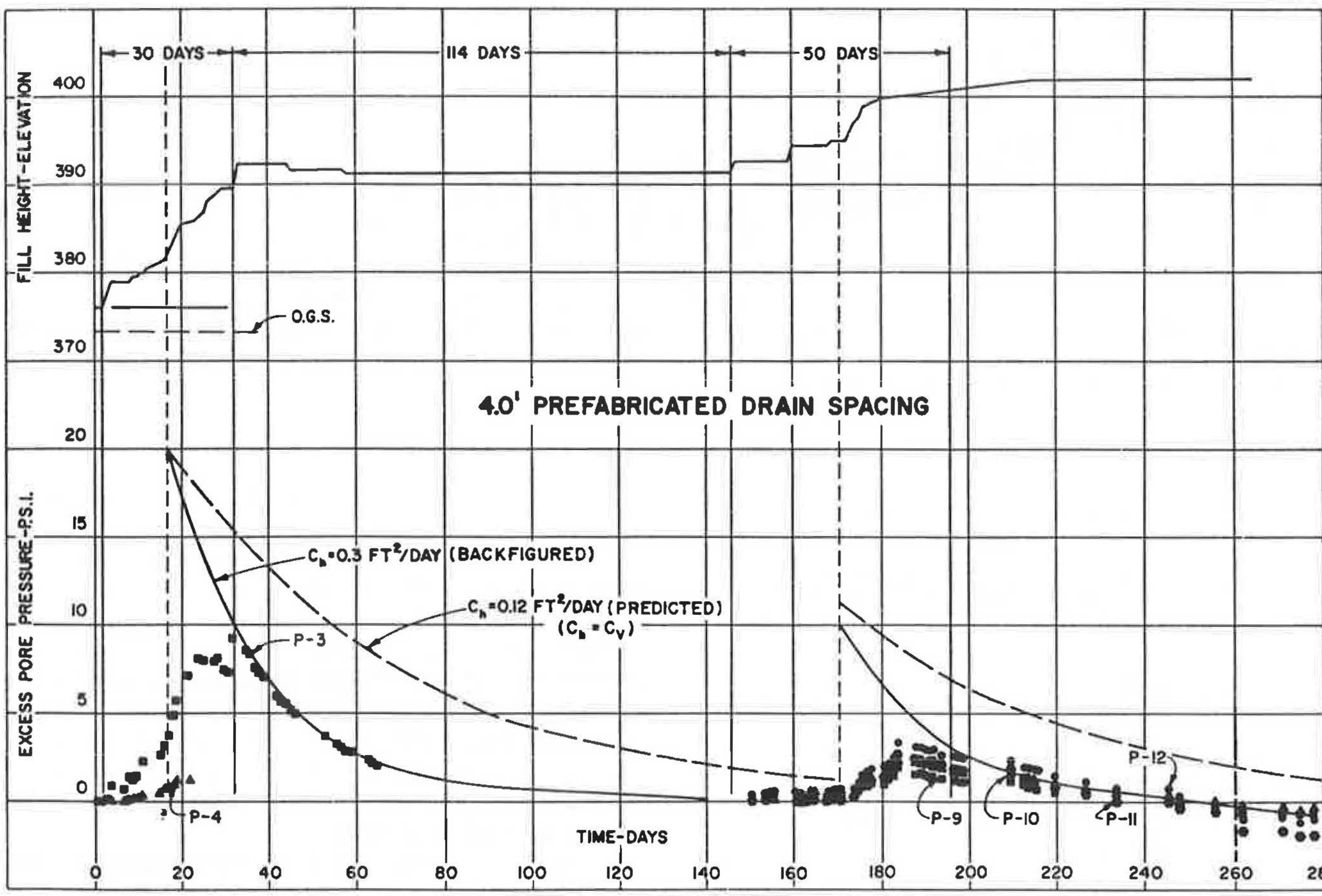

FIGURE 12 Predicted and measured excess pore pressure with 4.0-ft prefabricated drain spacing.

Second Stage

All of the piezometers ceased to function at the end of the 4-mo waiting period. Six new pneumatic piezometers were installed before placement of the second stage at the locations shown in Figure 10. Four piezometers were installed in the area of the 4.0-ft drain spacing and two piezometers in the area of the 7.5-ft drain spacing.

The second stage construction for the remainder of the fill was completed in approximately 50 days. An additional 0.5 to 1.0 ft of total settlement occurred in conjunction with the second stage loading (Figure 9). The excess pore pressures as shown in Figures 12 and 14 appeared to respond to the fill placement and dissipated with time as expected.

EVALUATION OF FIELD PERFORMANCE

Field Coefficient of Consolidation

A determination of the field coefficient of horizontal consolidation (c_h) was made based on the rate of dissipation of excess

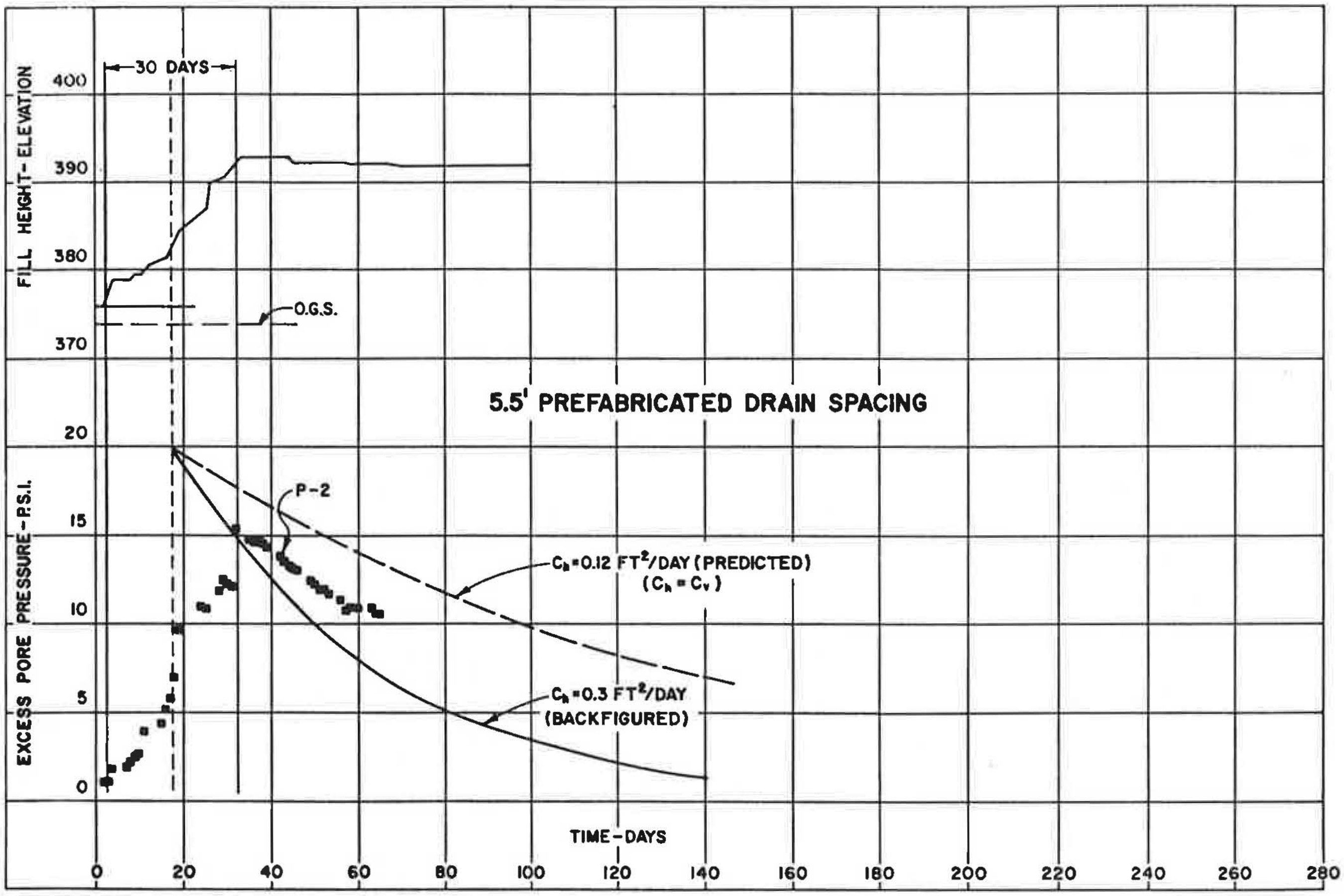

FIGURE 13 Predicted and measured excess pore pressure with 5.5-ft prefabricated drain spacing.

pore pressure recorded in the piezometer located in the 4.0-ft spacing area (Figure 12). A value of c_h = 0.3 ft^2 per day was determined based on the assumptions of radial drainage only and the piezometer being located in the center of the drainage grid. If the piezometers were actually located at the 1/4 points between drains, the value of c_h determined based on the assumption of the piezometer being equally spaced between drains would only be overestimated by a maximum factor of 10 percent (Figure 15). The value of c_h determined based on the assumption of the piezometer being located in the center of the drainage grid is, therefore, reasonable and corresponds well with the block permeability test results which indicated $c_h \approx 3c_v$.

Performance of Different Vertical Drain Spacings

The efficiency of the drain spacing based on an evaluation of the excess pore pressure dissipation is shown in Figure 16. This figure shows the recorded change in excess pore water pressure with increasing embankment pressure during embankment placement. In the area of the 4.0- and 5.5-ft drain spacings, excess pore pressures were dissipating during embankment construction. On completion of the first stage, approximately 50 and 75 percent, respectively, of the total anticipated excess pore pressure had dissipated. Although no measurable pore pressure dissipation was recorded where the drains were spaced at 7.5 ft, it would be expected that approximately 12 percent dissipation should have taken place based on c_h = 0.3 ft^2 per day. Twelve percent equates to approximately 2.4± psi pore pressure dissipation, which can be considered small enough to be nondefinitive in verifying the actual field measurement (i.e., $\Delta u/\Delta p$ = 0.88 in lieu of 1.0). It is also possible that the piezometer may have started to malfunction.

SUMMARY AND CONCLUSIONS

The design and performance of an approach embankment constructed over soft subsoils was investigated. Construction scheduling and weak soils necessitated a foundation treatment that consisted of prefabricated vertical drains and a two-stage embankment construction. In order to investigate the behavior of the embankment during construction, field measurements using settlement gauges and pneumatic piezometers were undertaken.

From the data presented herein and for the soils on this particular project, the following conclusions can be drawn.

- The results of field instrumentation indicated that prefabricated drains can be used with confidence in providing a cost effective means of accelerating foundation settlements and allowing for safe construction of embankments within a reasonable time period.
- Back analysis of the excess pore pressure dissipation rates indicated that the coefficient of horizontal consolidation (c_h) was approximately three times greater than what was assumed

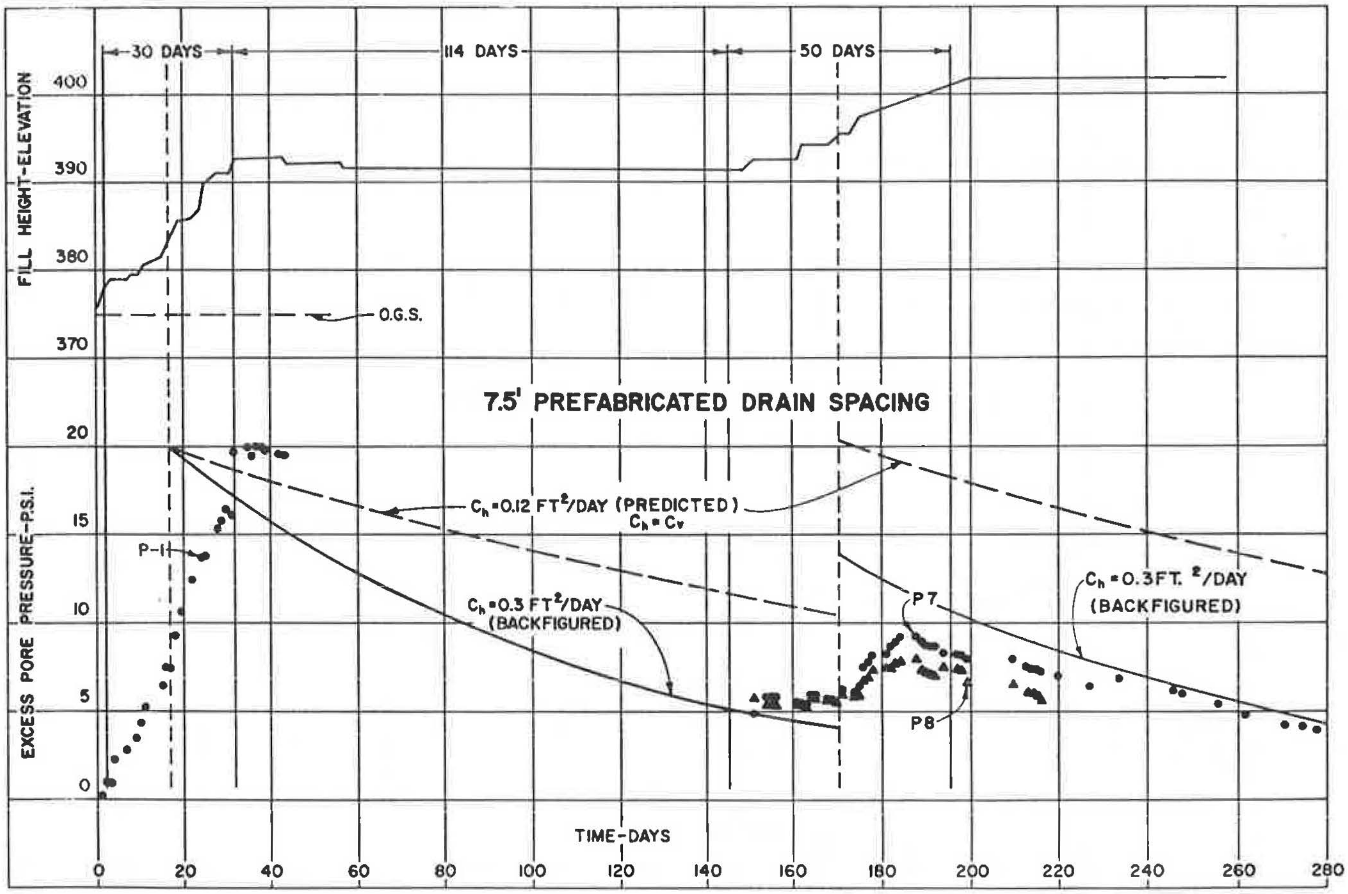

FIGURE 14 Predicted and measured excess pore pressure with 7.5-ft prefabricated drain spacing.

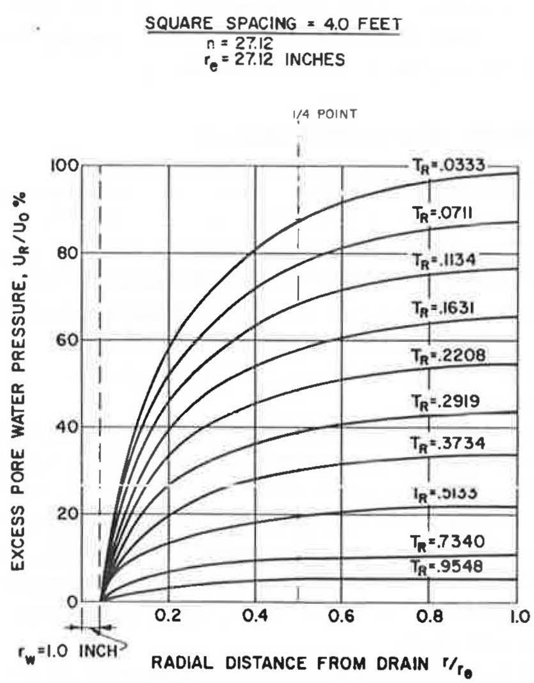

FIGURE 15 Excess pore water pressure versus radial distance from drain.

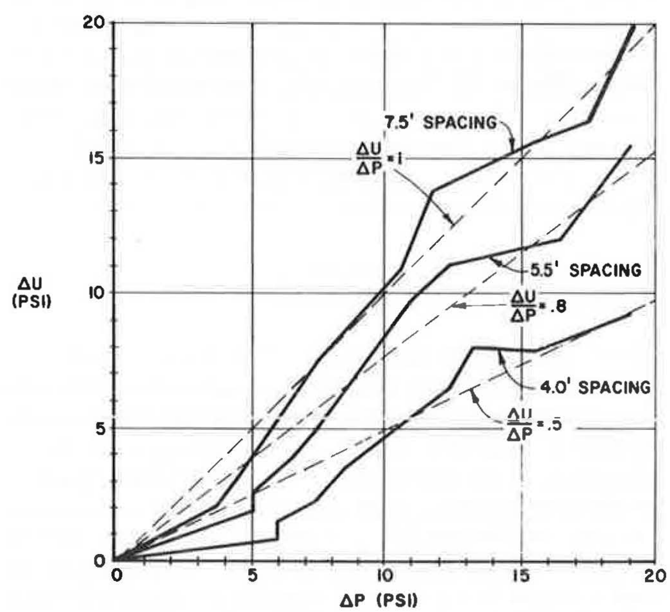

FIGURE 16 Piezometer response during embankment loading.

in design based on the assumption that the piezometers were equidistant between drains. If the piezometers were actually located at the 1/4 points between drains, less than 10 percent overestimation of c_h would occur. Therefore, this assumption of the piezometers being equidistant between drains is a reasonable one in the determination of c_h. The value of c_h obtained from piezometer data is in agreement with results obtained from laboratory block permeability tests on the clayey silt soil.

- Settlement predictions from one-dimensional consolidation tests agreed reasonably well with what was observed in the field.
- The drains were effective in reducing the required amount of excess pore pressure buildup during and after embankment construction.
- An equivalent sand drain diameter of 2 in. was considered as being an appropriate value to use in design for the Alidrain based on laboratory large-diameter consolidation tests and field performance from piezometer readings.
- The Barron-Kjellman formula for predicting time rates was considered to be an appropriate model.

REFERENCES

1. B. H. Fellenius. The Background and Theory of Vertical Drains with Particular References to the Alidrain. Notes for Lecture in Skelleftea, Montreal, Aug. 24, 1977.
2. L. D. Suits, R. L. Gemme, and J. J. Masi. The Effectiveness of Prefabricated Drains on the Laboratory Consolidation of Remolded Soils. *Consolidation of Soils: Testing and Evaluation* (R. N. Yong and F. C. Townsend, eds.), ASTM STP 892. American Society for Testing and Materials, Philadelphia, Pa., 1986, pp. 663–683.
3. T. A. Carlo. *Procedure for Determining the Ratio of the Horizontal to Vertical Coefficient of Consolidation by Means of the Block Permeability Test.* Soil Mechanics Bureau, New York State Department of Transportation, Sept. 1984.
4. C. C. Ladd and A. E. Z. Wissa. *Geology and Engineering Properties of Connecticut Valley Varved Clays with Special Reference to Embankment Construction.* Department of Civil Engineering Research Report R70-56, Soils Publication. No. 264, MIT, 1970.
5. M. Cregg. *Installation of Wick Drains, I-81, Hiawatha Boulevard to Mattydale, Contract 1, D96872.* Soil Mechanics Bureau, New York State Department of Transportation, March 1983.

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Experience with Wick Drains in Highway Construction over Soft Clay—Storz Expressway, Omaha, Nebraska

STEVEN R. SAYE, CHARLES N. EASTON, WAYNE D. SMITH, KENNETH H. NASS, AND CHARLES C. LADD

Wick drains were installed along portions of the 2-mi-long highway embankment crossing up to 50 ft of soft highly plastic clay deposited in a cutoff oxbow of the Missouri River in Omaha, Nebraska. About 1,650,000 lineal feet of 4-in.-wide band-type wick drains were installed in a triangular pattern at spacings ranging from 3.25 to 5.5 ft to facilitate strength gain of the foundation clays during stage construction and to precompress the clays under a surcharge before paving. The wick drains have allowed the excess pore water pressures generated by fill placement to dissipate. Apparent horizontal coefficients of consolidation calculated from pore pressure and settlement observations range from about one-half to several times the values estimated for design. Measurements during three consolidation intervals to date have shown that the wick drains continue to function without significant changes in the apparent horizontal coefficient of consolidation after vertical strains greater than 15 percent and total settlements greater than 5 ft. Instrumentation placed between the wick drains has shown that the apparent horizontal coefficient of consolidation is significantly smaller where the wick drain spacing is 3.25 ft than in similar soil layers where the spacing is 5.5 ft.

The Arthur C. Storz Expressway will be a 1.9-mi-long four-lane divided highway in Omaha, Nebraska. The alignment begins on a loess-mantled terrace overlooking the Missouri River and extends eastward over the flood plain. Thick deposits of soft high-plasticity clay underlie about 60 percent of the alignment. Fill heights of 12 to 30 ft were necessary to construct the permanent embankment and surcharges.

Due to the poor subsurface conditions identified along the alignment, a special foundation stabilization program was developed to satisfy stability requirements and postconstruction settlement criteria. The recommended stabilization program included use of vertical wick drains, stabilizing berms, surcharge fills, and staged embankment construction. Various instruments were installed to provide data with which to evaluate the embankment and foundation performance.

This paper summarizes the design and installation of the wick drains and the observed wick drain performance over a 3-yr period during staged construction.

S. R. Saye, C. N. Easton, and K. H. Nass, Woodward-Clyde Consultants, 10842 Old Mill Road, Suite 2, Omaha, Nebr. 68154. W. D. Smith, Woodward-Clyde Consultants, 5055 Antioch Road, Overland Park, Kans. 66203. C. C. Ladd, Department of Civil Engineering, MIT, Cambridge, Mass. 02139.

SITE CONDITIONS

The presence of thick, soft, high-plasticity clays and the requirement for significant embankment thicknesses resulted in critical stability conditions with significant settlements in two segments of the roadway alignment called the Bluff area and Florence Lake. The centerline embankment geometry and the soil conditions in these areas are summarized in Figures 1 and 2.

The vertical alignment design at the Bluff area required a large cut through the loess mantled terrace and placement of up to 30 ft of embankment over the recent flood plain soils. The subsurface profile at the juncture of the terrace and flood plain deposits consisted of a complex system of low- and high-plasticity clays, silts, and sands resulting from intermittent erosion of the loess terrace and periodic flooding of the Missouri River. This transitional zone ends about 200 ft from the toe of the bluff, and the flood plain deposits become primarily high-plasticity clays overlying a thin sand layer and limestone bedrock. This profile continues for about 1,300 ft with decreasing design embankment height from west to east.

Summarized in Figure 3 are the index properties and stress history within the alluvial soils near Station 202 near the juncture of the alluvial soils and the loess mantled terrace. These soils are highly layered and predominantly low plastic. The initial apparent stress history profile is shown to be complex, being strongly affected by the artesian water conditions at the rock surface and desiccation of surface soil layers. The grading program forced much of the profile into virgin consolidation.

The terrace sands that underlie the loess deposits at the Bluff extend into the flood plain beneath the upper alluvial soils. They form a confined aquifer with preconstruction piezometric water levels extending up to 11 ft above the floodplain surface. The piezometric elevation in the terrace sands appeared to decrease with increasing distance from the Bluff, greatly affecting the stress history and strength of the foundation soils.

Settlement and stability were also critical design issues in a clay-filled oxbow of the Missouri River referred to locally as Florence Lake. Figure 2 shows the centerline embankment geometry and subsurface profile along Florence Lake. Thick deposits of soft high-plasticity clay overlie thick alluvial sands beneath this 3,500-ft segment of the roadway. The maximum clay thickness approaches 50 ft. The index properties and stress history conditions in the Florence Lake oxbow near Station 265

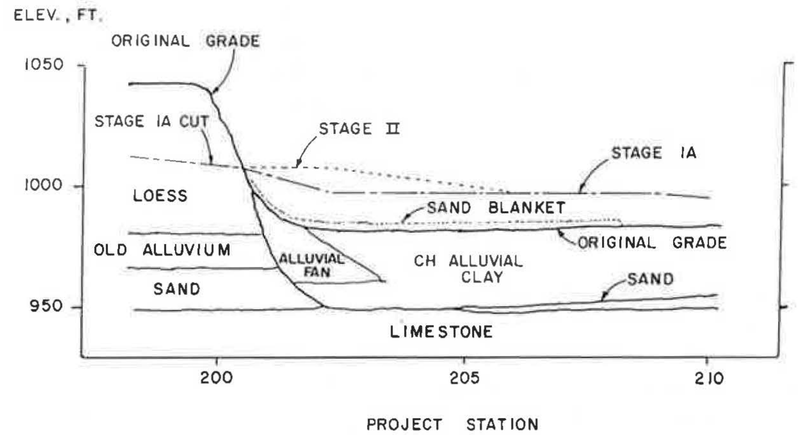

FIGURE 1 Simplified geologic section at Bluff area.

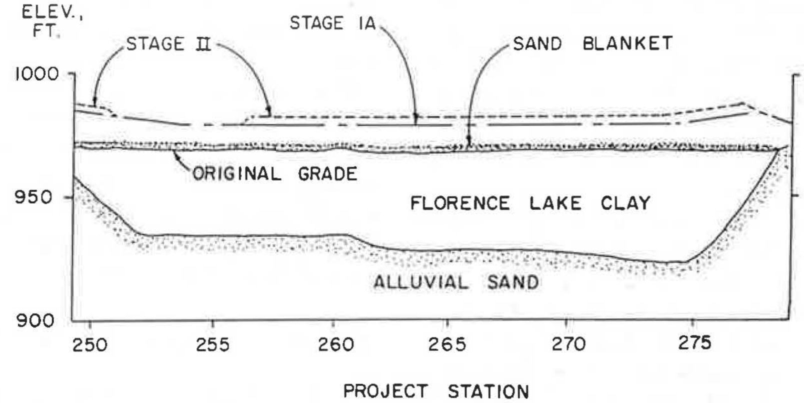

FIGURE 2 Simplified geologic section at Florence Lake.

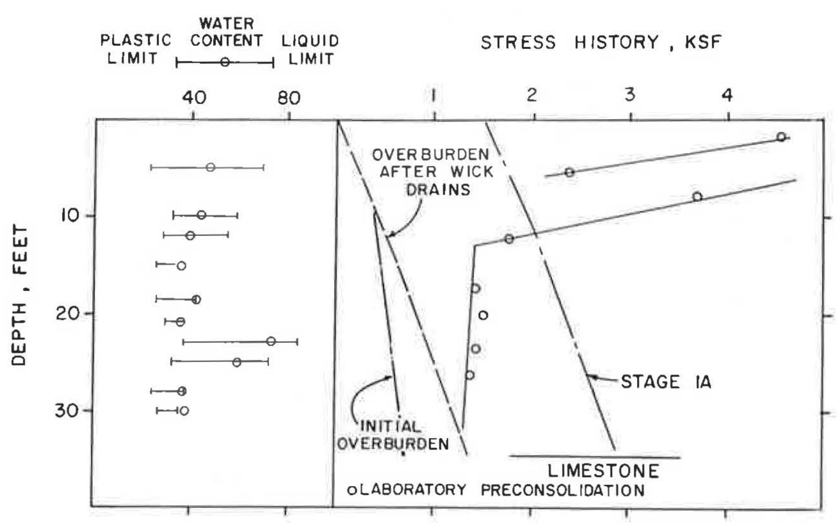

FIGURE 3 Soil conditions at Bluff area, Station 202.

are summarized in Figure 4. The upper 10± feet shows improved strength and precompression from desiccation. The lower clays appear to be nearly normally consolidated. Preconstruction piezometric water levels in the clays and underlying alluvial sands typically varied from about 0 to 4 ft below the ground surface.

EMBANKMENT DESIGN

The embankment design included a stabilization program for the Bluff and Florence Lake areas. The program made use of vertical wick drains, stabilizing berms, preloading with temporary surcharge fills, and staged embankment construction (*1*). The wick drains were to be installed under a pregrading contract beginning in December 1983, and ending in May 1984. The first stage of embankment construction (Stage IA) was scheduled to begin in the spring of 1984 and scheduled for completion in the fall of 1984. Where staged construction was required, the schedule provided for start and completion of the final stage of embankment construction (Stage II) in the spring and summer of 1985. Fine grading and the start of paving were scheduled to begin in the spring of 1987. This schedule allowed a consolidation period of 880 days for single-stage construction. For two-stage construction, consolidation periods of 300 and 550 days were provided for Stage IA and Stage II, respectively. The actual periods required to complete embankment construction were extended and Stage IA was actually completed over a 2-yr time period.

Prefabricated wick drains were selected instead of jetted nondisplacement sand drains based on economy and to accommodate the winter construction schedule. Design of the drain spacing, which varied from 3.25 to 10 ft, used an equivalent drain diameter of 0.22 ft (based on the diameter of a circle with the same perimeter as the wick drain) and a design horizontal coefficient of consolidation (C_h) of 0.05 ft^2 per day (Bluff area) and 0.025 ft^2 per day (Florence Lake) to achieve 90 percent consolidation within the construction schedule. These design C_h values were selected from the results of one-dimensional consolidation tests, an assumed ratio of horizontal to vertical coefficient of permeability of 2, and an assumed 50 percent reduction of C_h as a result of soil disturbance during drain installation. Drain spacing beneath temporary stability berms was based on a 50 percent degree of consolidation between stages.

INSTRUMENTATION

Instrumentation was installed to monitor stability and settlement during fill placement and subsequent consolidation intervals to provide a comparison of predicted and actual behavior and to provide a basis for adjustments in the design, if needed (*2*). Primary instruments to monitor consolidation of the foundation clays included Sondexes that measure relative settlements with depth and pneumatic piezometers at several depths to monitor excess pore water pressure. Inclinometers were used to measure lateral displacements within the foundation clays, both to help assess overall stability and to adjust centerline settlements for the effects of lateral deformation in conjunction with finite element analyses (*3–4*). Typical sections across the embankment and instrument locations are shown in Figures 5 and 6.

Fourteen Sondexes and seventy-three pneumatic piezometers were installed at selected stations along the alignment. These instruments were installed at the center of the triangular spacings before wick drain installation to allow observation of soil behavior during installation of the drains. Remarkably, only two Sondex instruments were damaged by the wick drain mandrel and all piezometers survived the drain installation.

SPECIFICATIONS AND CONSTRUCTION

The wick drains were installed under contract with the Nebraska Department of Roads. The wick drain contract included clearing and grubbing, placement of a 2- to 3-ft-thick

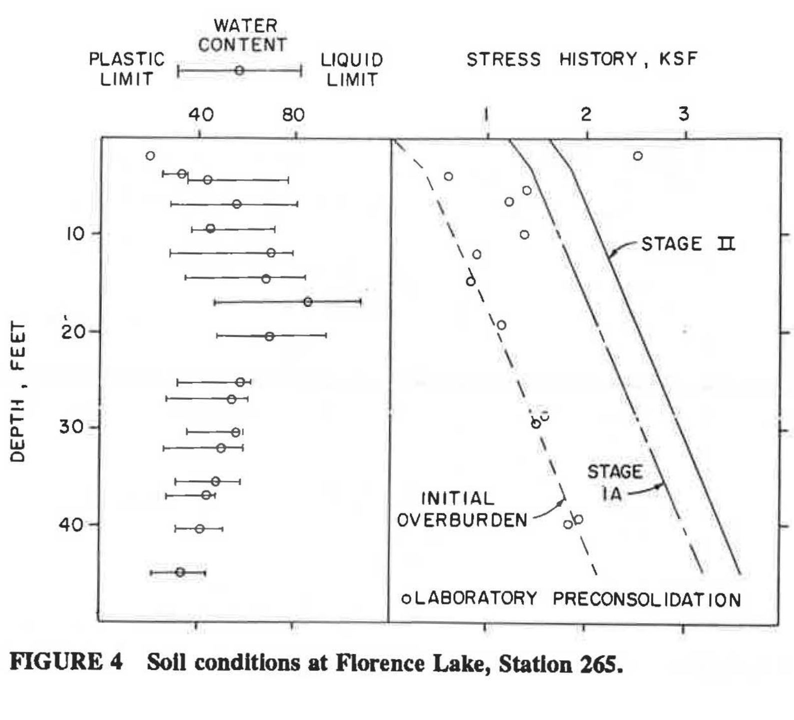

FIGURE 4 Soil conditions at Florence Lake, Station 265.

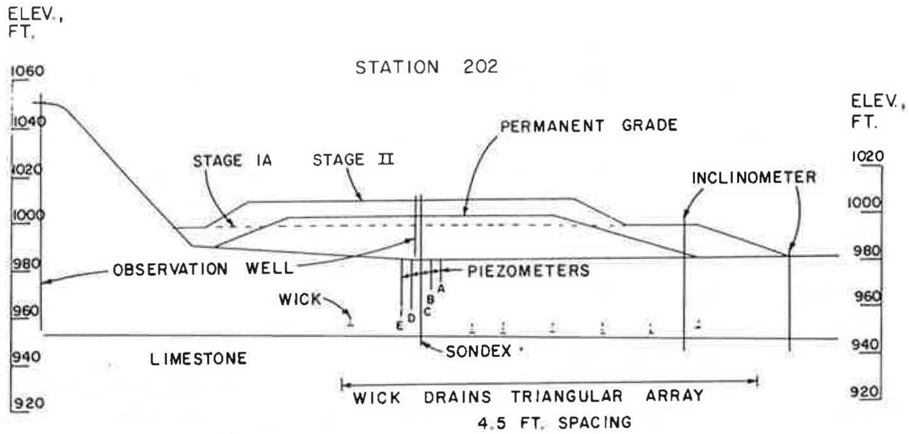

FIGURE 5 Typical embankment section and instrument locations at Bluff area, Station 202.

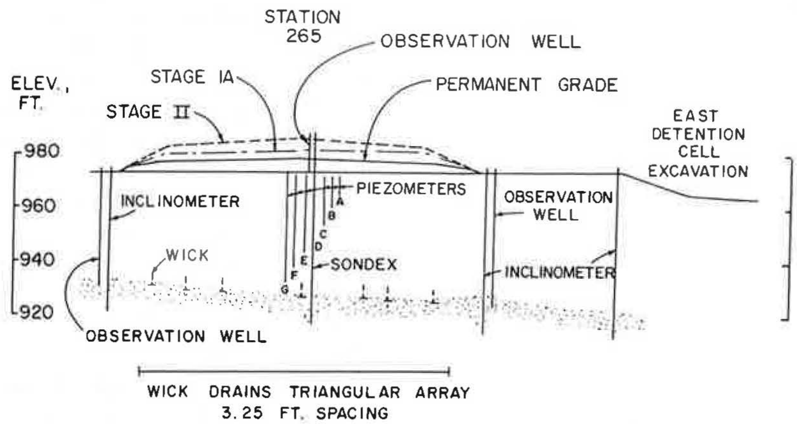

FIGURE 6 Typical embankment section and instrument locations at Florence Lake, Station 265.

sand blanket, installation of a subdrain collection system in the sand blanket, temporary drainage culverts, and wick drains.

Contract documents specified approved wick drain manufacturers. Wick drains were to be the type formed by a band-shaped plastic core which permits continuous vertical drainage wrapped in a separate filter fabric. Specifications called for drains to be installed using a hollow mandrel, with an anchor rod or plate to fix the base of the drain. The mandrel cross sectional area was limited to 10 sq in. to reduce disturbance of the clay. The contractor was required to demonstrate his equipment, methods, and materials by installing 10 trial drains at each of three to five locations designated by the engineer.

Although prequalification of potential contractors was considered during preparation of the wick drain contract, the final specifications did not require experienced specialty contractors. The local contractor that successfully completed the contract had no previous wick drain experience.

Wick drains were installed in a triangular spacing with a 3-in. position tolerance specified. The length of wick drains was documented during installation by observing the depth of penetration of the wick drain mandrel. Payment for the wick drains was made at a contract unit price per lineal foot of installed drain measured from the top elevation of the sand blanket to the tip elevation of the wick drain. Bid quantities for the wick drain contract included an estimated 1,564,840 lineal ft of wick drain.

Three firms submitted bids for the wick drain contract ranging from \$1.69 million to \$1.77 million. Unit prices for installation of wick drains ranged from \$0.33 to \$0.52 per lineal ft and formed about half of the total contract cost. Unit prices for the sand blanket and subdrain system ranged from \$13.37 to \$14.50/cu yd. About 850,000 cu yd of foundation soil were treated with the wick drains resulting in a unit construction cost of about \$2.00/cu yd of treated soil. The costs included mobilization, clearing and grubbing, sand blanket, underdrains, and wick drains. Klaasmeyer Brothers Construction Company of Omaha, Nebraska, completed this work from December 1983 to March 1984.

Initial construction included clearing and grubbing followed by placement of a 2- to 3-ft-thick sand blanket. The sand blanket provided a working surface for installation of the drains and embankment fill placement and provided drainage for water exiting the wick drains.

The contractor fabricated a cable-driven mandrel and leads supported by a crane as shown in Figure 7. Schedule requirements required a second unit for a 2-mo period. About, 40,800 Amerdrain™ Type 401 wick drains were installed with an average length of 40.5 ft. The contractor averaged 370 drains

(15,000 lineal feet) per rig per day through the project during wintertime installation of 1.65 million feet. Maximum installation rates approached 25,000 lineal ft per rig per day. Weather conditions were not a significant factor during drain installation except that work halted when daytime ambient air temperatures fell below 0°F. A backhoe with a pneumatic hammer (pavement breaker) was used periodically to break through frost in the sand blanket before drain installation. Inspection personnel for the wick drain contract included a project engineer and one technician for each rig. The inspector documented the position and length of the wick drains and rejected those not anchored at the base of the clay. Experience showed that a 6-in.-long $^3/_8$-in.-diameter reinforcing rod successfully anchored the drains after about 0.5 ft of penetration into the alluvial sands.

FIGURE 7 Wick drain installation equipment.

OBSERVATIONS OF WICK DRAIN PERFORMANCE

The records of settlement and piezometric elevation with respect to time from the instrumentation provided the basis for evaluation of the effectiveness of the wick drains in accelerating the rate of consolidation of the foundation clays.

Bluff Area

The instrumentation observations at instrument cluster IC202 (Figure 5) located in the Bluff area illustrate the effectiveness of wick drains installed at a 4.5-ft triangular spacing in highly layered soils. The soils in the transition between the loess terrace and the flood plain consist of low- and high-plasticity silty clays interbedded with numerous horizontal sand and silt layers underlain by alluvial sands that transmit artesian water conditions. Figure 8 is a plot of the piezometric elevation and fill height at Instrument Cluster IC202 with respect to time. Very small excess pore water pressures (measured piezometric elevation minus reference piezometric elevation) developed in the foundation clays during filling. The data suggest that the numerous sand and silt layers were effectively dewatered by the wick drains, which greatly improved the vertical drainage conditions in addition to the horizontal flow. Figure 8 also shows a 5± ft drop in the piezometric elevation in the terrace sands following wick drain installation. Visual observation of the subdrains and sand blanket has shown continuing water flow over the past 4 yr. The gradual increase in the piezometric elevation in the terrace sand with time suggests that the drains are progressively becoming less effective in dewatering the higher piezometric elevation. The lower piezometers in the clay do not show a similar increase suggesting that the drains continue to function. The gradual decrease in the piezometric elevation in the sand blanket appears to be due to gradual settlement of the outlets of the subdrain system.

The relationship between embankment loading and settlement at Instrument Cluster IC202 shown in Figure 9 indicates that most of the settlement occurred during loading, which is consistent with the piezometer data. About 2 ft of settlement has occurred under about 28 ft of fill. Further from the Bluff, the sand and silt layers end, and the instruments show slower consolidation consistent with radial drainage in thick clay. These observations are not discussed.

Florence Lake

The observations at Instrument Cluster IC265 (Figure 6) located near the center of Florence Lake provide a representative record of the consolidation behavior of the thick, soft, high-plasticity oxbow clays with wick drains arranged in a triangular pattern and spaced at 3.25 ft. Summarized in Figures 10 and 11 are the observations of piezometric elevation and settlement with respect to time. Before Stage II, a settlement of about 4.2 ft occurred in response to an effective fill thickness (actual thickness minus one-half observed settlement) of 11.5 ft. Measurements of settlement with depth in Figure 12 show maximum strains greater than 15 percent, with an average strain of about 10 percent.

The piezometer data show development of large excess pore water pressures during each of three filling periods (Figure 10), followed by slow rates of dissipation, as would be expected for loads applied to a thick deposit of normally consolidated clay.

Installation of the wick drains in the Florence Lake deposits caused significant excess pore water development as illustrated in Figure 10 by the increased piezometric water elevations recorded at Station 265.

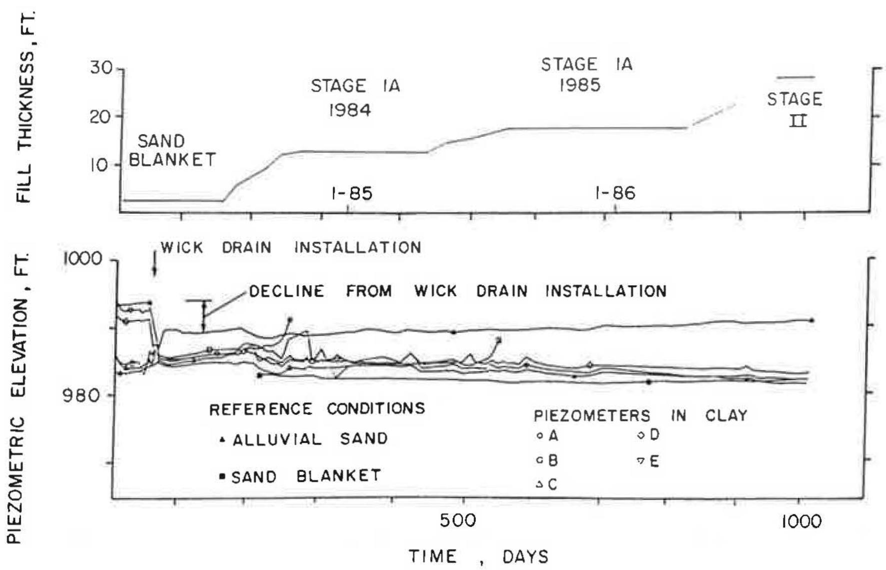

FIGURE 8 Fill thickness and piezometric elevation versus time at Bluff area, Station 202.

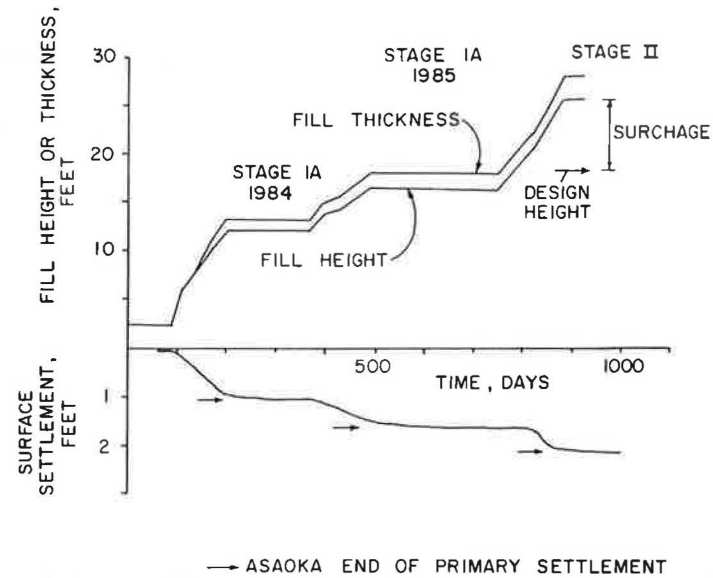

FIGURE 9 Fill thickness and surface settlement versus time at Bluff area, Station 202.

Figure 13 is a plot of the measured increased heads versus depth at five Florence Lake locations with wick drain spacings of 3.25, 5.25, and 5.5 ft. The rather large scatter in the data probably results in part from variations in the actual (versus intended) horizontal distance between the piezometers and the wick drains. The data indicate generally larger heads (4 ± 2 ft) within the normally consolidated clay below 20 ft than at shallower depths (2 ± 1 ft). Somewhat higher values were noted at the three locations with the closest drain spacing.

The rates of pore water pressure dissipation and settlement measured during the consolidation periods after each of the three loadings were analyzed to back calculate (C_h) for the foundation clays. Analyses of the piezometer data used the method developed by Orleach (*5*), as illustrated in Figure 14, which is a plot of the excess piezometric head on a log scale versus natural time. The data should form a straight line for a constant C_h, negligible effects of vertical drainage, and constant load. The slope of this line should give a unique C_h independent of the plan location of the piezometer tip relative to the vertical drains.

Analysis of the settlement data used the Asaoka (*6*) method illustrated in Figure 15. This technique predicts the total settlement at the end of primary consolidation. C_h can be calculated from the slope of the linear portions of the plotted data, again assuming a constant C_h, negligible vertical drainage, and constant load.

Figure 16 is a plot of values of C_h obtained from piezometer and observation well readings during the 1985 consolidation period at five Florence Lake locations. For wick drains spaced at 3.25 ft, most of the data fall within a fairly narrow range with a mean value about one-half of the C_h = 0.025 ft^2 per day value selected for design. In contrast, data from areas with

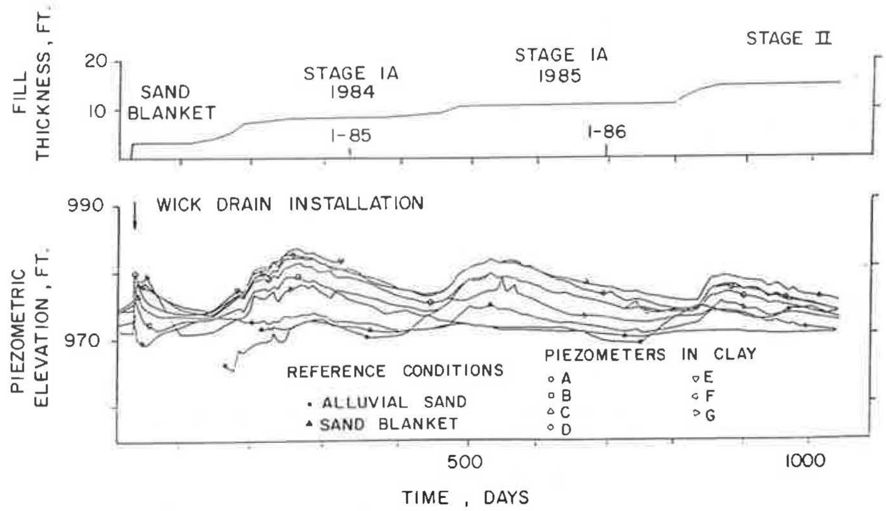

FIGURE 10 Fill thickness and piezometric elevation versus time at Florence Lake, Station 265.

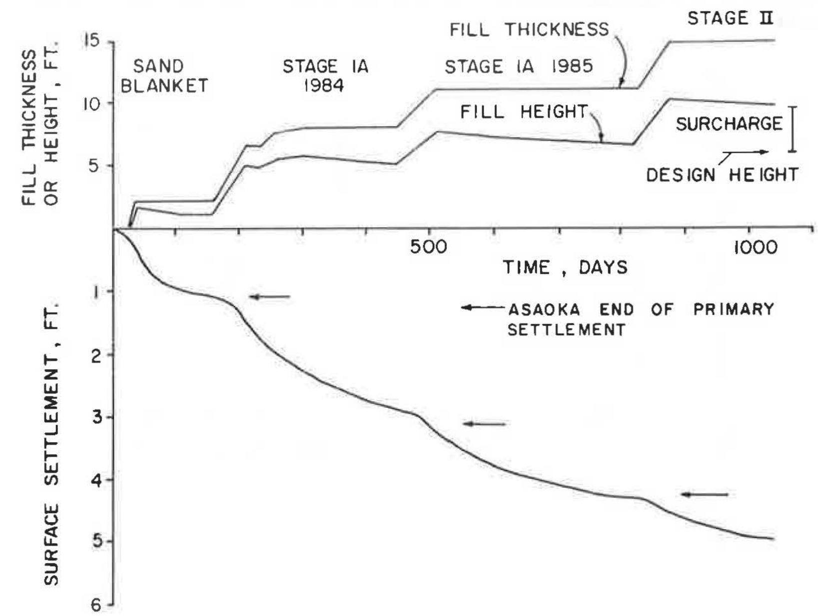

FIGURE 11 Fill thickness and surface settlement versus time at Florence Lake, Station 265.

larger drain spacings yield C_h values scattered about the design C_h except near the bottom of the clays. Vertical drainage near the bottom of the clay may have contributed to pore pressure dissipation resulting in misleading values of C_h determined using the Orleach (5) analysis. The significantly lower C_h at the closest spacing is attributed to increased disturbance of the soft clay during wick drain installation.

The above results and the data shown in Figure 13 suggest an overall degree of soil disturbance from wick drain installation that also may have caused a reduction in the initial undrained strength of the Florence Lake soft clays. Larger than predicted lateral deformations recorded by inclinometers during the 1984 Stage IA embankment construction led to a decision to stop fill placement because of foundation stability concerns. Whether the less stable conditions occurred because of disturbance or variations in the design stress history, undrained strength parameters, or a combination thereof remains uncertain.

DISCUSSION OF WICK DRAIN PERFORMANCE

The design C_h values resulted in a 3.25-ft triangular drain spacing to achieve 90 percent consolidation in the available time period for stage construction within most of Florence Lake. The actual C_h values at this spacing, as measured by the instrumentation, were significantly lower than the design values resulting in a longer time interval for primary consolidation than predicted. It was not necessary to change the construction schedule because fill placement also progressed more slowly than planned. The first increment of fill (Stage IA, Figures 2, 6, and 10) was not completed until the fall of 1985 resulting in an unscheduled consolidation interval during the

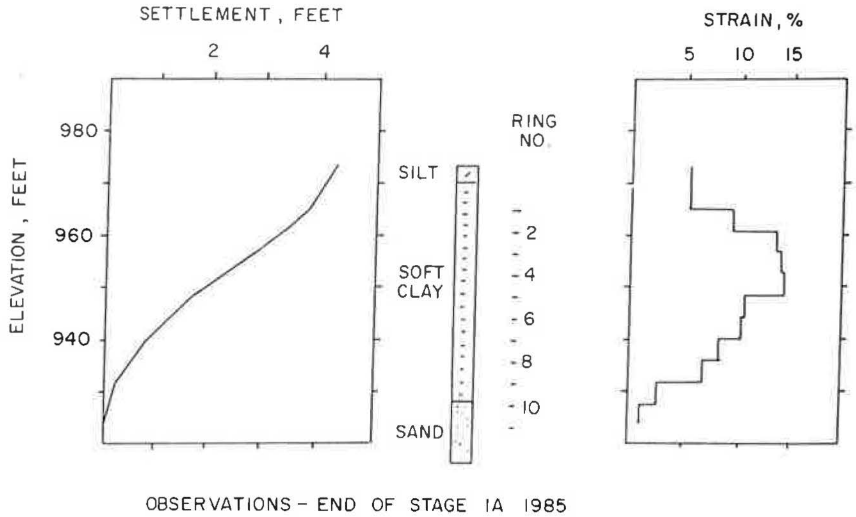

FIGURE 12 Distribution of settlement and strain versus depth at Florence Lake, Station 265.

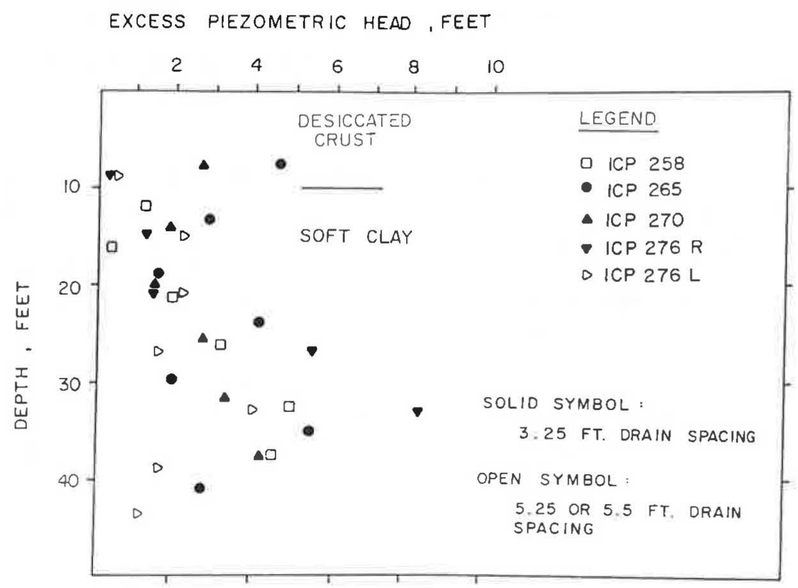

FIGURE 13 Excess piezometric head caused by wick drain installation at Florence Lake.

winter of 1984 to 1985. This prolonged schedule allowed sufficient consolidation to occur before placement of the Stage II fill.

Drain spacing designed to accommodate the C_h values determined from field measurements and the original time schedule would have required decreasing the minimum drain spacing to about 2.7 ft. It is surmised that the smaller drain spacing probably would have caused a further reduction in C_h and possibly more severe stability problems during initial filling due to increased disturbance of the soft clay from wick drain installation.

As previously noted, observation wells were installed in the sand blanket and in the deep alluvial sands beneath the clay to monitor the ambient piezometric elevation necessary for interpretation of the piezometer data. The water level in the deep alluvial sand at Florence Lake varies 4 to 6 ft in an annual cycle related to the navigation season of the Missouri River. In summer, the piezometric elevation rises to near the original ground surface. During this period the drain system discharge increases as water flows from the alluvial sand through the wick drains to the sand blanket and into the subdrain system. The water level in the sand blanket remains relatively constant, matching the invert of the subdrain system. This water flow indicates that the longitudinal transmissivity of the wick drains has not been damaged by the large strains and that the wick drains are in good hydraulic contact with the alluvial sands. In winter, the piezometric elevation in the alluvial sand drops to near or below the subdrain outlet elevation, and the flow from the subdrain system stops.

Based on measurements of the amount and rate of primary settlement following Stage IA filling, the degree of consolidation before paving was estimated and the amount of required surcharge recalculated. Based on these data, the limits and thickness of the Stage II fill were either confirmed or changed.

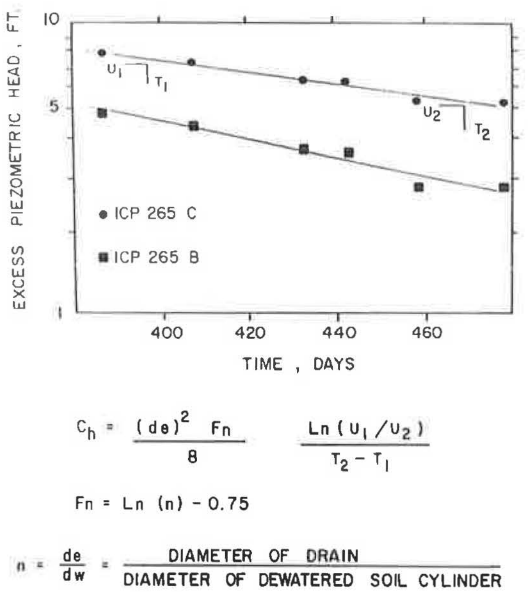

FIGURE 14 Orleach evaluation of C_h, Station 265.

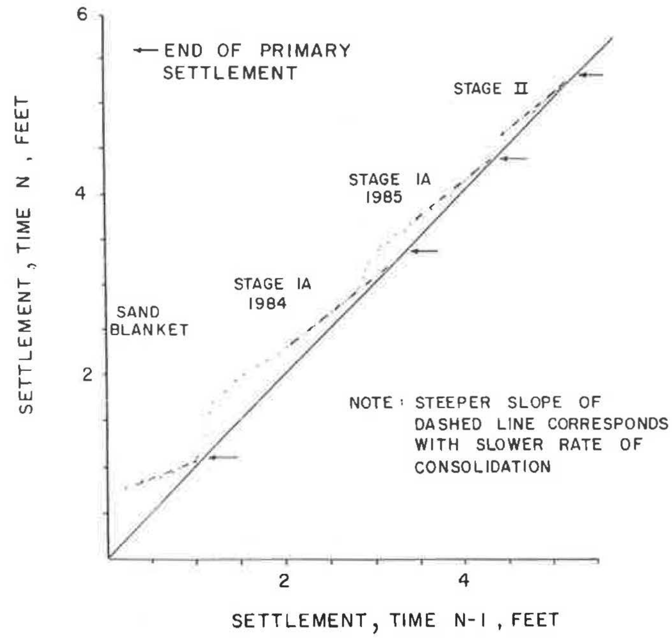

FIGURE 15 Asaoka projection of primary settlement, Station 265.

CONCLUSIONS

Analysis of the field settlement and piezometer data shows that the wick drains significantly reduced the time necessary for primary consolidation of the foundation clays along the Storz Expressway alignment. The wick drains continued to function over a 3-year time period with total settlements of over 4 ft and strains exceeding 15 percent. Significant changes in C_h during consolidation were not noted.

The field observations confirm that installation of the wick drains disturbed the soft Florence Lake clays. Where wicks were installed at a 3.25-ft triangular spacing, the effective C_h values were about 50 percent lower than used for design and about 50 percent lower than where drains were installed at 5.25- and 5.5-ft spacings. Uncertainties in the effects of disturbance from drain installation greatly complicate the selection of drain spacings for foundation stabilization projects with soft clays and a fixed construction schedule. Conservative design estimates of C_h appear justified in determining wick drain spacings.

Where the wick drains intercepted horizontal layers of high permeability, such as existed at the Bluff area, the effective drainage conditions were greatly improved, leading to field rates of consolidation much faster than predicted.

A comprehensive instrumentation system should be an important part of the overall design process for staged

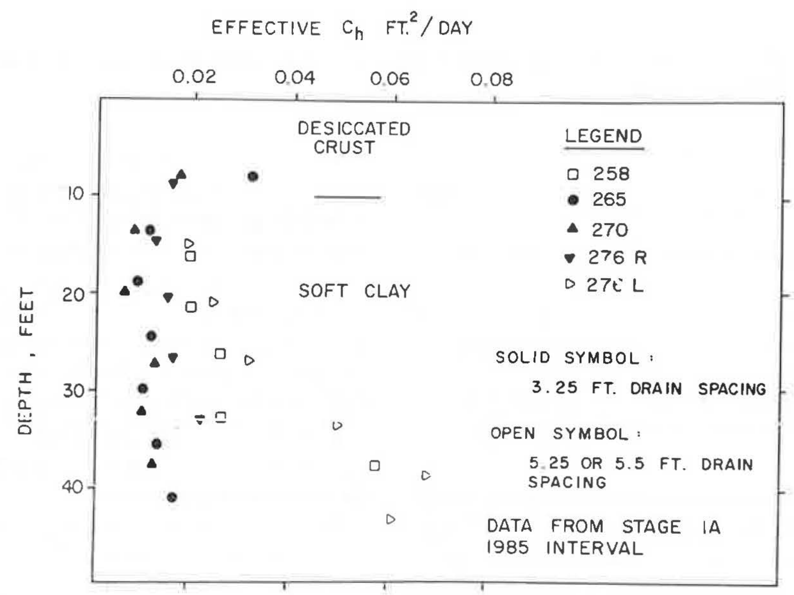

FIGURE 16 Comparison of C_h from different wick drain spacings at Florence Lake based on Orleach analysis of piezometer data.

construction of embankments over soft clays. This project showed that instruments can be successfully installed with drain spacings as small as 3.25 ft without excessive damage during drain installation.

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REFERENCES

1. C. C. Ladd. *Use of Precompression and Vertical Drains for Stabilization of Foundation Soils.* P76-4. Department of Civil Engineering, MIT, Cambridge, Mass., 1976.
2. C. N. Easton. Instrumentation of Embankment on Soft Clay. *Professional Development Symposium.* Woodward-Clyde Consultants, St. Louis, Mo., 1986.
3. R. J. N. Navarro-Banzer. *Predicted Performance of a Soft Clay Foundation During Initial Loading.* S.M. thesis. Department of Civil Engineering, MIT, Cambridge, Mass., 1984.
4. E. H. M. Rady. *Performance of a Soft Clay Foundation During the Staged Construction of an Embankment.* S.M. thesis. Department of Civil Engineering, MIT, Cambridge, Mass., 1985.
5. P. Orleach. *Techniques To Evaluate The Field Performance of Vertical Drains.* M.S. thesis. Department of Civil Engineering, MIT, Cambridge, Mass., 1983.
6. A. Asaoka. Observational Procedure of Settlement Prediction. *Soils and Foundations.* Vol. 18, No. 4, 1978, pp. 87–100.

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