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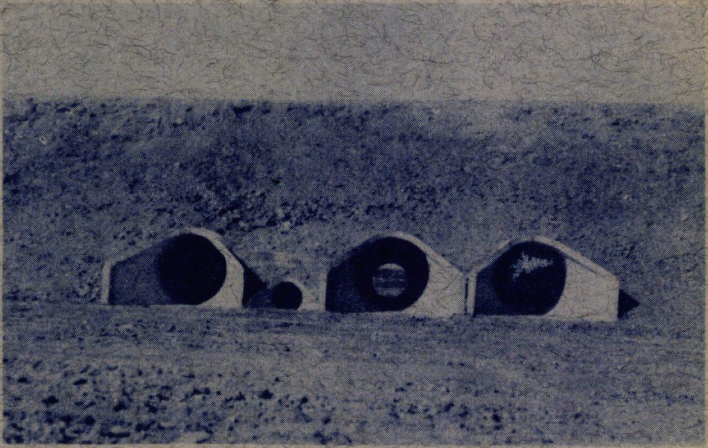

60-INCH PIPE JACKED IN PLACE   WATERTOWN, SOUTH DAKOTA

Soils Committee Reports and Special Papers

1948

PRESENTED AT THE
TWENTY-SEVENTH ANNUAL MEETING

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HIGHWAY RESEARCH BOARD

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SOILS COMMITTEE REPORTS
AND
SPECIAL PAPERS

PRESENTED AT THE TWENTY-SEVENTH ANNUAL MEETING

1947

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH
NATIONAL RESEARCH COUNCIL

Washington 25, D.C. October, 1948

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DEPARTMENT OF SOILS

C. A. HOGENTOGLER, *Chairman*

REPORT OF COMMITTEE ON FROST HEAVE AND FROST ACTION IN SOIL

K. B. WOODS, *Chairman*, CHARLES W. ALLEN, EARL F. BENNETT, J. E. LAWRENCE, A. E. MATTHEWS, FRANK R. OLMSTEAD, H. R. SMITH, J. H. SWANBERG

SYNOPSIS

The present Project Committee on Frost Heave and Frost Action in Soil was constituted in June, 1947. Its scope is intended to cover all phases of frost action in soils. It is an outgrowth of a committee on "Freezeproofing Treatment of Soils and Calcium Chloride" which was originally organized in the fall of 1943 to study the use of calcium chloride in preventing detrimental frost heave in subgrades. The original committee on freezeproofing treatment of soils with calcium chloride is being continued as a subcommittee under the new project committee.

This report is a summary of the findings of the original committee. The scope of the original committee was twofold, namely; (1) to study the use of calcium chloride as an admixture with soils for minimizing frost heave; and (2) the use of calcium chloride mixed with subgrade soils to minimize frost action and the resulting frost damage to pavements.

There has been much laboratory work performed in this field during the past several years and the available data indicate strongly that calcium chloride mixed with soil will minimize or eliminate frost heave when the calcium chloride is used in sufficient quantities and when the chemicals are not leached from the soil under conditions of adverse water flow.

Field installations of calcium chloride mixed or added to subgrade soils have been made on projects in Minnesota, Indiana, Michigan, and Massachusetts in areas where severe frost heave or frost damage has been noted. The results of these field experiments have been erratic. In several of the installations no apparent benefit resulted from use of the chemical to minimize the frost heave. The data collected by the committee indicate that in some of these cases, adverse water conditions may have been responsible, in part, for the frost heave; likewise, these adverse water conditions may have been largely responsible for leaching the soluble chemical from the treated subgrade.

As a result of these largely negative results, the committee feels that it is not worthwhile to continue the field experiments to eliminate differential frost heave; however, a program is being developed to determine the effectiveness of the use of calcium chloride in subgrades and base courses for the purpose of minimizing the amount of break-up which occurs on some roads during the spring of the year because of saturation of the subgrade during the melting period.

The Sub-Committee on Freezeproofing Treatment of Soil with Calcium Chloride was appointed in the fall of 1943 for the purpose of evaluating the practicability of treating subgrades with calcium chloride to eliminate frost heave, frost action, and frost damage. The committee has had five meetings since it was appointed and a number of field installations have been made in Minnesota, Michigan, Indiana, and Massachusetts. Most of the field installations to date have been made for the purpose of treating subgrades which heave periodically. Most of the highway projects were surfaced with portland cement concrete. The method of treatment, in most cases, consisted of drilling holes through the pavement, removing soil from beneath the holes, and filling the spaces with calcium chloride or with calcium chloride brine. Wherever possible similar sections were left untreated so that a comparison could be made between treated and untreated sections.

MINNESOTA, J. H. Swanberg

T. H. No. 12 - 2.8 Miles East of Willmar

In a shallow cut a series of sharp heaves occurred annually within a length of about 400 feet. The vertical movement amounted to about 0.5 foot in the most severe heave. A section, 160 feet in length, was treated in August, 1944. Holes spaced 3 feet apart were dug from each edge of the slab with 6-inch post hole augers. These holes extended beneath the slab to within one foot of the center of the pavement and were inclined slightly downward toward the center of slab, the bottom of the holes being from one to one and one-half feet below the bottom of slab. Commercial calcium chloride was then placed in each hole at the rate of about 80 lb. per hole, using a pipe and a rammer to force the calcium chloride into the holes. Some water was poured in to dissolve the chloride. A total of 8500 lb. of calcium chloride was used, which was equivalent to slightly less than 27 lb. per sq. yd. of surface. Soils in the subgrade varied, but silt loam pockets in medium to fine sand apparently were the cause of the more severe heaving conditions. Ground water level was more than 6 feet below the bottom of the slab.

A frost proof bench mark was placed and profile levels were taken on center line and 6 feet right and left of center line in August, 1944; January 19, 1945; March 7, 1945; January 16, 1946; March 18, 1946; and on February 12, 1947.

These winter profiles indicate very little, if any, decrease in the amount of heaving within the treated section.

T. H. No. 61 - 3.2 Miles South of Rush City

A series of six sharp heaves occurred every year in a cut 400 feet in length. The soils in this area consist of a sand mantle of variable depth overlying fine sandy loam and clay loam tills. The grade line cuts through the sand mantle in this cut, and at the crest of each heave, the plastic soils are found directly beneath the pavement. At the low points between the crests of the heaves, medium to fine sand is found for some depth below the pavement slab. A 200 foot section of treatment was placed to include the most severe heaves which had a maximum vertical rise of about 0.4 feet.

Holes were drilled vertically through the slab using pneumatic drills, with a 2-5/8 inch star bit. The holes were drilled on 3-foot centers starting 1-1/2 feet from the edge of the 18-foot slab. Below the slab the holes were enlarged with soil augers and air jets so that a hole from 6 inches to 8 inches in diameter and from 12 inches to 18 inches below the bottom of slab was formed. A total 9000 pounds of commercial calcium chloride was placed in these holes, an average of about 22.5 lb. per sq. yd. Water was poured in to dissolve the chloride after which the holes were filled with sand. This treatment was completed in September, 1944.

Profile levels were taken over this treatment on October 4, 1944, January 25, 1945 and March 12, 1945. These levels indicated no reduction in heaving in the treated section.

In October, 1945, additional chloride was placed in a 15 foot section of this treatment at the crest of the most severe heave. On the left half of the slab, 400 pounds of chloride was added and on the right half of the slab 1200 pounds of chloride was added. In this treatment the chloride was first dissolved, and the solution poured into the holes over a period of two weeks.

Profile levels were taken over this treatment on January 17, 1946, and on March 14, 1946. These levels showed a reduction in heave over the 15 foot section having the retreatment of from 1 to 1-1/2 inches, but a heave of from 2 to 3 inches still existed.

The cost of the original treatment on this section was $1.35 per square yard.

CONCLUSIONS

From the data obtained, it appeared that these treatments were ineffective. Prints showing the details of these treatments and profiles have been previously submitted to the committee.

INDIANA, R. E. Frost

Route U. S. No. 30, in northern Indiana, has several sections of dual lane pavement which were constructed in 1936 and 1937, in which it was necessary to make some deep cuts. Some severe frost heaves developed in several of these deep cuts on sect ons of this road in Porter County just south of Valparaiso. In the fall of 1944 one of these heaves was treated with calcium chloride to see whether or not the heave could be minimized.

The section of the pavement, which was treated, was in the north lane between Stations 1157 ≁ 98 and 1158 ≁ 38. The heave area in the south lane was left untreated so that the use of calcium chloride in the north lane could be evaluated. A total of 1800 pounds of calcium chloride was applied to the subgrade of this cut section through holes drilled into the concrete pavement and in a trench along the south edge of the pavement. Flake calcium chloride was used to fill each hole which had been drilled through the portland cement concrete pavement and then water was poured into the hole to dissolve the chemical. The procedure was repeated until the desired quantity of calcium chloride had been applied.

Soil samples were taken from several of the holes drilled through the portland cement concrete pavement at 12, 24, and 30-inch depths. In addition, six three-inch galvanized iron downspout pipes were installed for making ground-water observations.

In the early spring of 1945 detailed surveys were made on the treated and untreated sections of this road. It was found that the heave which had been treated was raised a maximum of 0.18 feet as compared with a maximum of 0.68 feet in other adjacent, untreated sections of the north lane. The maximum heave in the south lane in a comparable situation to the frost heave of the north lane was 0.20 feet. From this it was concluded that the salt had not stopped the heave, but that it may have minimized the total amount somewhat. However, the ground-water records indicate that there is a transverse flow of water through this cut which probably leached the chemical from the soil.

MASSACHUSETTS, J. E. Lawrence

In the spring and fall of 1944 several projects in Massachusetts were treated experimentally with calcium chloride to determine the effectiveness of the chemical in reducing frost heave. One of these roads was on Auto Route 127 in District No. 5, Town of Beverly, The road was a bituminous macadam and the treatment was between Station 1100 and 1100/20. Calcium chloride was applied to 2 1/2-inch diameter holes treated to a depth of two feet. A total of nine pounds per square yard was applied in this fashion. In the fall of 1944 additional chemicals totaling 18.7 pounds per square yard were applied to this experimental section. Observations made in the spring of 1945 indicated that the treatments were somewhat effective in minimizing frost heave. During the winter of 1945-1946 the frost heaving was reduced to a maximum of 2 inches with slight break up in the macadam surface.

In District 4, Town of Harvard, Auto Route 111, some experiments were initiated in the spring of 1944 between Station 6900 / 45. The pavement was a sand and gravel tar surface. The calcium chloride was applied in flake form through 2 1/2-inch diameter holes, two feet on center, to a depth of eighteen inches below the surface of the road. The south half of a road only was treated with a total of four pounds per square yard of calcium chloride. In October, 1944, eighteen additional pounds per square yard was applied to this section. In the spring of 1945 observations were made and it was concluded that the calcium chloride had been effective in minimizing the amount of heave. During the winter of 1945-1946, the frost heaving was greatly reduced, but the road surface was so badly damaged and broken up that it was necessary to rebuild the surface.

In District No. 5, Town of North Andover, Auto Route 114, a severe heave was treated at Station 16200 / 30. Chemicals were applied in 1942, 1943, and 1944. It is known that the road heaved at this location to as much as ten inches each year. Following the years of application of chemical, the heave was entirely eliminated. During the winter of 1945-1946 no noticeable heaving was observed and no cracking of the concrete surface resulted. This road is now being reconstructed. A summary of the experiments show that while there was a reduction in the heaving due to the use of calcium chloride there was surface damage resulting from the chloride treatment.

MICHIGAN, A. E. Matthews

In order to determine the effectiveness of treating subgrade soils with calcium chloride for the prevention of detrimental frost action, four locations were selected in Michigan. Following is a brief description of the work at these locations:

Stanwood Location
US-31, Mecosta County, Approximately 500 Feet North of the Village of Stanwood

This heave occurred in concrete pavement. It is 50 feet in length. The maximum amount of heave is 3 inches. The entire heave was treated in September, 1944. Holes (2-5/8" diameter) were drilled through the pavement on 3-foot centers. A combination of air jet and auger was used to enlarge the holes in the subgrade to 6 inches in diameter and to 12 inches in depth. Thirteen pounds of calcium chloride in solution form was added to each hole. After the solution had soaked away the holes were filled with gravel and covered with cold patch material. Soil profiles were determined by auger borings along the outside edges of the slab (10' R. & L. of C/L). The subgrade soil is variable but the heave was caused mainly by very fine sand and silt with some clay.

A frost proof bench mark was established. Profile levels were taken along the outside edge of the slabs on March 14, 1944, November 1, 1944, January 23, 1945, and February 26, 1945.

The treatment was ineffective in reducing the amount of heaving.

Gladwin Location
M-61, Gladwin County, Approximately 1-1/2 Miles West of the Village of Gladwin

This heave is in a bituminous surface treated gravel road. It is only 10 feet in length. The maximum amount of heave is 1-1/4 inches. The heave was treated in September, 1944. Holes on 3 foot centers were dug through the road surface. These were extended with a 6 inch post hole auger to a depth of 12 inches below the gravel. Thirteen pounds of calcium chloride in solution form was added to each hole. After the solution had soaked away the holes were filled with gravel and capped with cold patch material.

Profiles of the subgrade soil were determined by auger borings 12' R. & 12' L. of C/L. The heave was caused by a small pocket of very fine sand and silt with some clay.

Profile levels (Fig. 1) were taken at center and along each edge of the surface on March 14, 1944, September 1, 1944, January 19, 1945, and March 6, 1945.

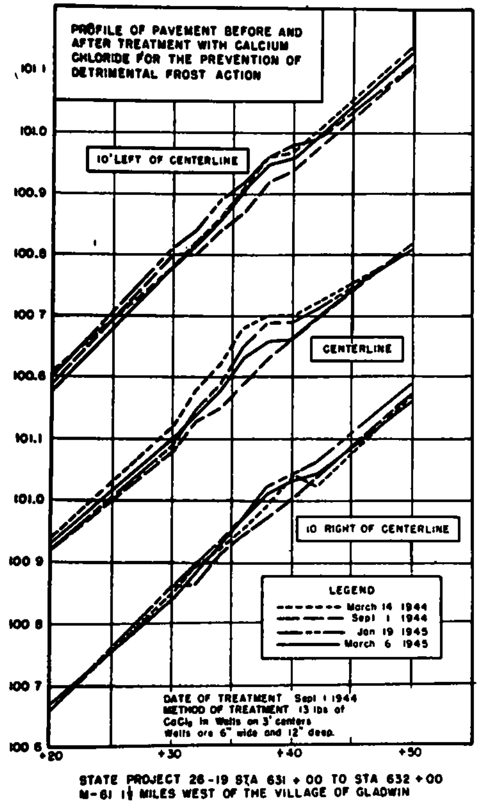

Figure 1. Profile Plot of a Pavement Before and After Treatment with Calcium Chloride.

The treatment was ineffective in reducing the amount of heaving.

Saginaw West Location
M-47, Saginaw County, Approximately 1/4 Mile South of Junction with US-10

This slight heave is in a concrete pavement. It extends beyond the limits of our 27-foot treatment. The old pavement was removed in August, 1944, on a patching contract. The right half of the subgrade at this time was treated as follows: Holes 6 inches in diameter and 12 inches in depth were dug in the subgrade on 3-foot centers. Thirteen pounds of calcium chloride (flake form) was added to each hole.

The soil profile 10' R. of ℄ was determined by auger borings along the edge of the slab. The subgrade soil is variable but the heave was caused mainly by very fine sand and silt.

Profile levels were taken at center and 9' R. & L. of center on November 21, 1944, January 19, 1945, and March 6, 1945.

A comparison was made between the treated section and the adjacent untreated section. The treatment was ineffective in reducing the amount of heaving.

Brighton North Location
US-23, 3.8 miles North of Junction with US-16 Near Brighton, Livingston County

This heave occurs in a concrete pavement. It is approximately 35 feet in length. The maximum amount of heave is 2.5 inches. The east half of this heave was treated in September, 1945. Holes (6" diameter) were drilled through the pavement on 3 foot centers. Holes in the subgrade were enlarged to 8 inches in diameter and 12 to 15 inches in depth. Twenty-five pounds of calcium chloride (solution form) were added to each on September 13, October 3, and November 1, 1945. The holes were then filled with gravel and covered with cold patch material.

Profiles of the subgrade soil were determined by auger borings along each edge of the slab (10' R. & L. of ℄). The soil is variable but the heave is caused mainly by very fine sand and silt.

Profiles levels were taken 5' R. & L. of ℄ at 10' intervals on September 7, 1945, February 25, 1946, and May 22, 1946. By comparing the treated section with the adjacent untreated section, we note that the treatment was ineffective in reducing the amount of heaving. Soil samples were taken during May, 1946, to determine the extent of migration of calcium chloride. From the test data, we note that the desired migration of calcium chloride has not taken place or that a greater part of the material has leached away.

CONCLUSIONS

From the information assembled in this work, we conclude that calcium chloride has not been effective in reducing frost heaving and that the desired migration of calcium chloride has not taken place or that a greater part of the material has leached away.

PURDUE UNIVERSITY, F. O. Slate

Our experiments cover "Permanence of Calcium Chloride in Subgrade and Stabilized Bases and Its Effectiveness on Base Course Densities."

Calcium Chloride is mixed into the base course of a road primarily to obtain and maintain a more or less favorable moisture content and, as a result, obtain a relatively high density. The purpose of this work was to determine the base course densities of several bituminous roads thus treated, after several years of use, and to determine how much of the calcium chloride originally placed there still remains.

Two samplings have been made--the first in the summer of 1942 and the last in the summer of 1946--to determine the changes of density and chloride content with time. Information on the original conditions of density and chloride content of the roads is in some cases so meager and unreliable that conclusions as to changes from the original conditions of the roads cannot be made. However, changes from the time of first sampling to that of the second have been measured and are reported.

Five roads are sampled: Hiwassee Dam Access Road, N. C., built in 1938; Dietrich-Montrose Road, Illinois, 1937; Muscatine County Road C, Iowa, 1939; Lawton-Marcellus Road, Michigan, 1938-39; and Kuttawa-Fredonia Road, Ky., 1941. Base course densities were measured by the sand method. Samples of base-course and subgrade material were analyzed for hygroscopic water content, and for chloride content

TABLE 1

DENSITIES OF BASE COURSES TREATED WITH CALCIUM CHLORIDE

Road	Year Sampled	Hole	Reported $CaCl_2$ used	Dry Density	Moisture Content	True Sp. Gr.	Maximum Theoretical Dry Density [a]
			lb. per sq. yd.	lb. per sq. yd.	%		lb. per cu. ft.
Hiwassee	1942	1	2.7	151 [a]	2.4	2.75 [c]	161
"	1942	2	2.7	144 [b]	2.6	2.75 [c]	160
"	1942	3	2.7	147 [b]	2.5	2.75 [c]	161
"	1946	1	2.7	150	2.3	2.70	158
"	1946	2	2.7	142	1.9	2.76	163
"	1946	3	2.7	141	2.3	2.80	164
Dietrich	1942	1	1.7	159 [d]	4.0	2.76 [c]	155
"	1942	2	1.7	147	3.3	2.76 [c]	158
"	1942	3	1.7	155	1.8	2.76 [c]	164
"	1946	1	1.7	163 [d]	4.6	2.76	153
"	1946	2	1.7	148	3.4	2.80	159
"	1946	3	1.7	137	2.8	2.72	158
Muscatine	1942	1	1.5	152	2.8	2.68 [c]	156
"	1942	2	1.5	153	2.8	2.68 [c]	156
"	1942	3	1.5	156 [d]	3.0	2.68 [c]	155
"	1946	1	1.5	134	3.5	2.70	154
"	1946	2	1.5	132	2.3	2.65	154
"	1946	3	1.5	145	1.7	2.70	161
Lawton	1942	1	1.1	146	5.0	2.80 [c]	153
"	1942	2	1.1	146	4.3	2.80 [c]	156
"	1942	3	1.1	145	4.5	2.80 [c]	155
"	1946	1	1.1	151	4.5	2.79	155
"	1946	2	1.1	146	2.2	2.77	162
"	1946	3	1.1	143	3.4	2.84	162
Kuttawa	1946	1	2.7	146	5.8	2.70	146
"	1946	2	2.7	137	6 7	2.71	145
"	1946	3	2.7	143	5 7	2.71	147

[a] Formula used: vol. soil (cu.ft) + vol. water (cu. ft) = 1 cu. ft. (assuming complete saturation)
let x = dry wt. (lb.); since vol. given as 1 cu. ft.
x = dry density (lb. per cu. ft.)

$$\frac{x}{(\text{true sp. gr.})\,(62.4)} + \frac{(\%\ \text{M.C.})\ x}{(100)\ \ (62.4)} = 1;$$

$$x = \frac{[(6240)\ (\text{true sp. gr.})]}{100 + (\%\ \text{M.C.})\ (\text{true sp. gr.})}$$

[b] Determined by Bernard Thomas.
[c] Average of 1946 values for same road.
[d] Impossible value; experimental error.

by the Mohr titration method. Chloride values from roadside blanks were subtracted from the amounts found under the pavements.

Table 1 lists the densities of the base courses sampled. The values labelled "Maximum Theoretical Dry-Density at M. C. Found" were calculated to determine whether the density values found are possible values (most of the values are so high, approaching those of concrete that doubt might be expresed as to their possibility).

Figure 2 shows the calcium chloride contents of the base courses and subgrades, calculated from the chloride found. Nothing is known of the calcium ion, or the manner in which the chloride is present, except that it is water-soluble. All depths represent distances down from the bottom of the pavement. Values of calcium chloride are expressed as the dihydrate, the commercial form in which the chemical is usually used in road work. All percentages are based on the dry weight of the soil.

The horizontal lines across the graphs represent the total calcium chloride applied (as reported), calculated as though it were mixed evenly throughout the 36 inches below the pavement, including base course and subgrade. In these calculations, an average density of 100 pounds per cubic foot was assumed for the entire 36 inches. In the Muscatine graph it is obvious that more chemical was used than was reported, perhaps for dust control in the years preceding pavement.

In a few years the 1946 values are slightly higher than the 1942 values. This might be caused by changes in ground-water conditions since the vertical migration of chloride is sensitive to rise and fall of water, but the difference, are too small to be significant in any case.

The density values found are unusually high, but there are no blank values (from untreated sections) for comparison. After a period of five to ten years, one-third to one-half of the chloride originally placed still remained. Almost the entire loss occurred during the first five years.

COMMITTEE SUMMARY

The work of the committee to date has been confined largely to the experimental treatment of existing frost heaves with calcium chloride. Figures 3 to 8, which follow, show typical procedures and records. Results have not been particularly encouraging since, in most instances, there has been no diminishing of the original amount of the heave. In contrast, however, some reports indicate some benefit and, in one instance, a complete cure. Since laboratory experiments have indicated that a moderate concentration of calcium chloride in the soil will practically eliminate frost heave, it is the committee's opinion, which is based in part on experimental data, that adverse water conditions have leached the chemical from the soil thus making the treatments ineffective in some instances.

The committee has not expanded its activities to include the treatment of subgrades in general, for the purposes of seeing whether or not the frost line can be lowered in the subgrade and thus decrease the amount of pavement spring break-up.

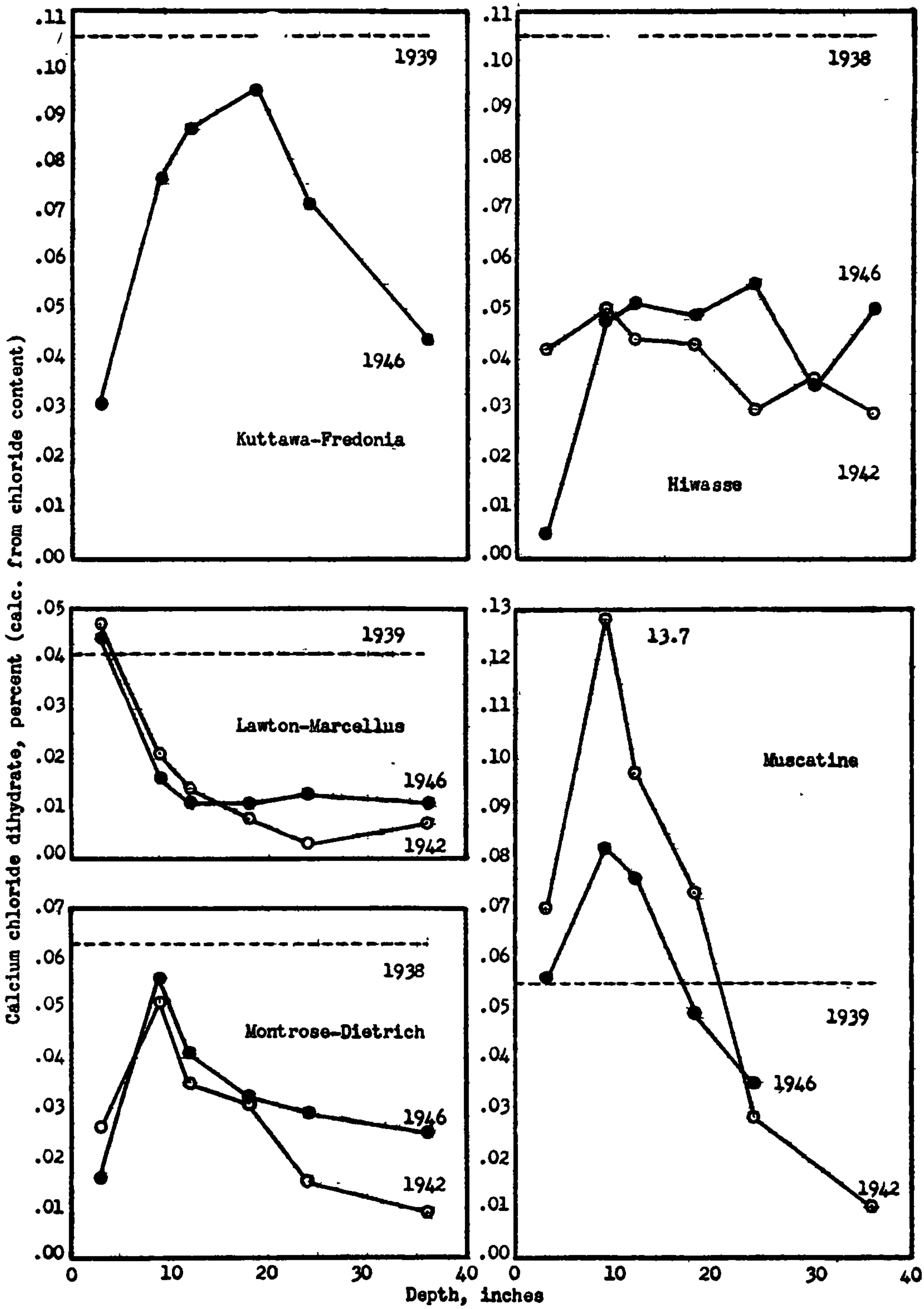

Figure 2. Calcium Chloride Contents of Subgrades and Base Courses of Several Roads. (Hort. dotted lines indicate amount of chemical reported originally used).

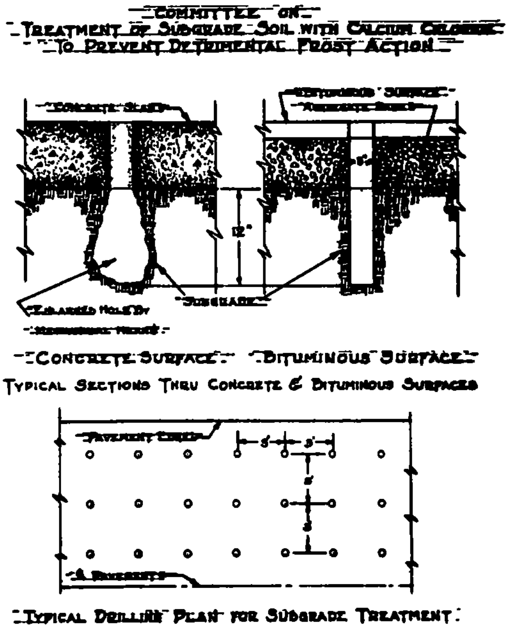

Figure 3. Plan and Cross-Section of Road, Showing Preparation for Application of Calcium Chloride.

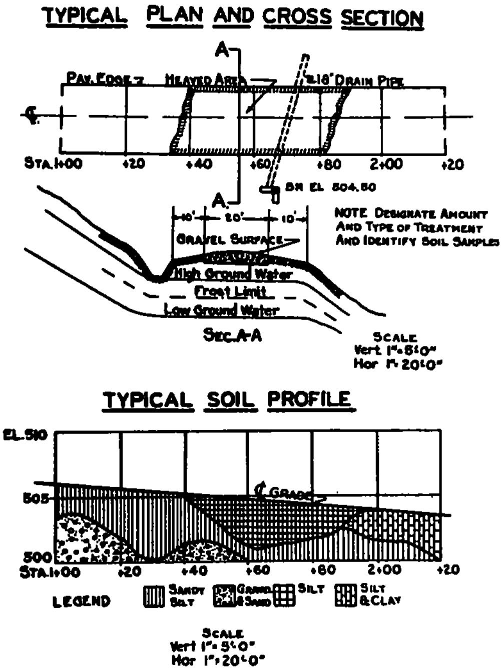

Figure 4. Subgrade Soil Profile and Cross-Section of a Road Treated with Calcium Chloride.

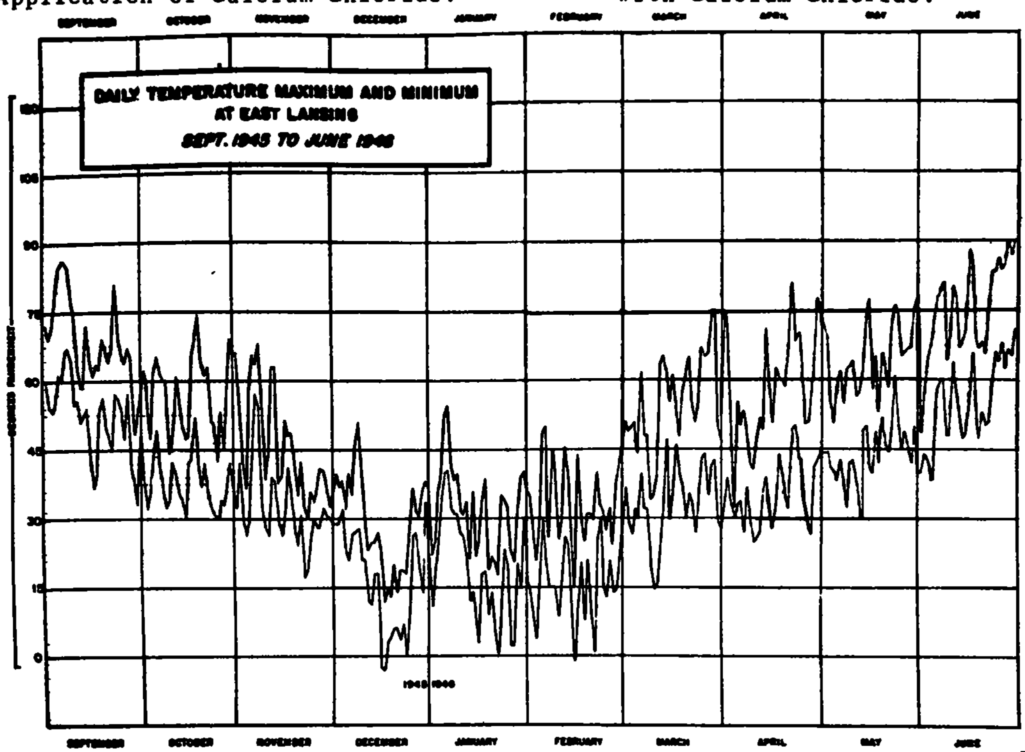

Figure 5. Typical Temperature Record Used for Frost Action Studies.

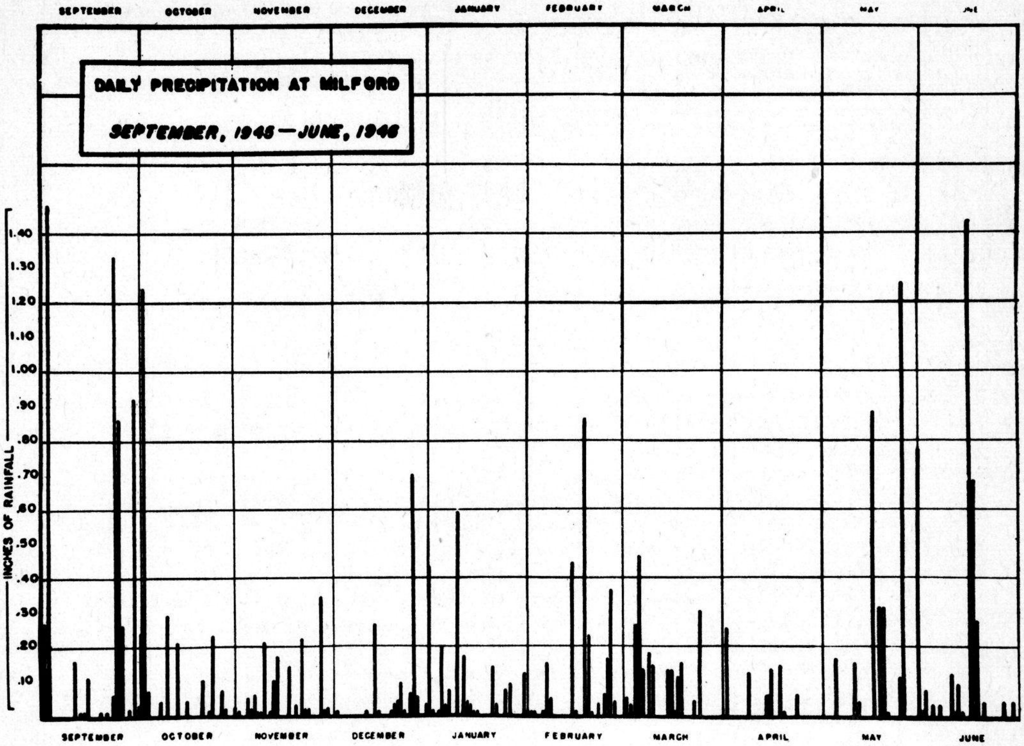

Figure 6. Typical Precipitation Record Used for Frost Action Studies.

Figure 7. Addition of Calcium Chloride and Water to the Subgrade of a Highway.

Figure 8. Calcium Chloride Being Poured Into a Trench Alongside a Pavement.

REPORT OF COMMITTEE ON SOIL CEMENT ROADS

CARL R. REID, *Chairman,* HAROLD ALLEN, M. D. CATTON, L. D. HICKS, C. A. HOGENTAGLER, JR., W. S. HOUSEL, T. A. MIDDLEBROOKS, F. V. REAGEL, T. E. STANTON, E. A. WILLIS, HANS F. WINTERKORN

SYNOPSIS

This report reviews the maintenance costs and conditions in service of soil-cement roads, as inspected and reported to the committee by the several states.

The construction details, age, traffic and present condition of the soil-cement base of these projects provides a complete history.

Many variables in the construction, climate and traffic indicate that six inches of soil-cement base properly constructed is entirely adequate for the purpose.

The committee desires that records of the condition of these and other earlier soil-cement projects be continued by the states and that subsequent studies be made.

Other uses of soil-cement mixtures and cement-modified soil binders for soil aggregate mixtures were considered by the committee and further studies recommended.

In 1945 the Committee on Soil-Cement Roads resumed activities. At the suggestion of the committee, the chairman circularized State Highway Departments to obtain maintenance cost data on the older soil-cement roads in service. The roads selected were those included in the 1940 committee report on Condition Survey of Soil-Cement Roads, reported on pages 812 to 820 inclusive of the Proceedings of the Twentieth Annual Meeting of the Highway Research Board. The reports from the various states replying were received quite late in the year. As a result, available data was discussed at the committee meeting held early in 1946 at Oklahoma City and reporting of the data deferred until more of the states could return the questionnaires.

The data requested on maintenance cost consisted of a year by year tabulation of maintenance expenditures for the soil-cement base, maintenance expenditures for the bituminous surface, and total maintenance for the project. This would permit calculation of the maintenance cost per mile per year of the soil-cement base, the bituminous surface, and the total. Data was also requested on the average traffic volume per day of commercial vehicles and passenger cars. In addition, it was requested that a rating be made of the present overall condition of the project.

A study of all data reported revealed that many variables entered into the base cost. With respect to bituminous surfacing it was impractical, if not impossible, to tabulate the returns for satisfactory comparison and this information was therefore, not included in Table No. 1 showing maintenance costs. Further, for the sake of simplicity, the data was reduced to show age of project, daily traffic, total soil-cement base maintenance

TABLE NO. I
MAINTENANCE COSTS OF SOIL-CEMENT ROADS (1945-6)

(Carl R. Reid)

State	No. (1)	County	Length Miles	Project Age Years	Daily Traffic Commercial	Daily Traffic Total	Soil-Cement Base Maintenance Cost Dollars Total	Soil-Cement Base Maintenance Cost Dollars Year/Mile	Present Condition (3)	Remarks
(1940 Surv.)										
Ga.	1	Clarke	2.00	8	-	-	0	0	E	
Ill.	1	Logan-Menard	2.9	6	90	185	0	0	E	
Ill.	2	Winnebago	1.14	9	-	40	525	10	G	130 ft. rd. repaired in 1940 & 43. Breakage due to heavy truck loads & excess moisture in subgrade.
Ken.	1	Stafford	1.23	7	155	1120	0	0	E	
Ky.	1	Daviess	6.54	7	180	450	0	0	E	6 small base failures have occurred-probably due to underlying clay stratum.
Md.	1	Worchester	2.65	7	24	46	0	0	E	
Md.	2	Carroll	1.1	6	41	163	0	0	E	
Miss.	1	Chickasaw & Pontotoc	23.78	6	115	637*	0	0		*1942 traffic
Ohio	1	Franklin	2.9	6	8	50	400	24	G	
Okla.	1	Caddo	5.80	5.5	200	535	2580*	81	G	*Includes fill settlement & surface.
N.C.	1	Beaufort	1.5	7	-	-	0	0	E	
"	2	Alamance	2.4	6	-	-	0	0	E	
N.C.	3	Wake	2.1	7	-	-	0	0	E	
"	4	Carteret	1.06	8	-	-	0	0	E	
Pa.	1	Lebanon	1.97	8	25	177	0	0	AV.	
Va.	1	York	2.75	6	500	2500	0	0	E	
"	2	New Kent	1.96	7	126	518	0	0	G	Maint. Cost for 1st 3 yrs. not available & not included.
Wash.	1	Whitman	1.00	7	25	120	0	0	E	
(2) Addnl. Proj.)										
Ga.		Liberty	1.24	3	-	200	0	0	E	
Ill.		Tazewell	3.86	4	-	500*	0	0	E	*1941 traffic
Md.		Queen Annes	2.16	4	29	144	0	0	E	
Md.		Queen Annes	2.31	4	19	66	0	0	E	
"		Dorchester	2.82	4	20	80	0	0	E	
Va.		Accomack	3.00	4	56	187	0	0	G	
Va.		Powhaton	1.00	4	159	719	0	0	E	
"		Sussex	2.49	4	80	400	391	39	G	
Wash.		Spokane	3.58	4	55	255	0	0	E	2" road mix surf. placed 1942. Other maintenance has been extremely small.

(1) Corresponding number used in 1940 survey report (20th Proceedings HRB).
(2) Additional projects reported since 1940 survey on which data were available.
(3) E - Excellent G-Good AV. - Average

cost during its life, the corresponding maintenance cost per mile per year and the reported condition or rating of the project.

The maintenance cost of the soil-cement base, Table No. 1, shows that the 1940 survey projects had an age varying from 5½ to 9 years. Eighteen projects are reported and of these there had been no maintenance expenditure on the soil-cement on 15 of them. On the remaining three projects, the base maintenance cost per mile per year was $10., $24., and $81. Twelve of the projects had a present condition rating of Excellent, four of them had a rating of Good, and one a rating of Average.

Also included in the tabulation are nine projects built since the 1940 survey. The age at time of reporting was four years, except for a three year job in Georgia. Only one job had any soil-cement base maintenance cost reported, and it had an average maintenance cost of $39.00 per mile per year. Seven of the jobs were rated Excellent and two were rated Good.

Traffic data was not submitted for all jobs, but of those reported, the commercial traffic varied from a minimum of eight trucks per day to a maximum of 500 trucks per day, and total traffic varied from a minimum of 40 vehicles per day to a maximum of 2500 vehicles per day.

Comments with returns by many states indicated maintenance cost records precluded separation of annual or total charges for only the base or surface course, or the actual outlay for the roadbed. also, in many instances two or more types of improvement were included in the mileage or section covered by the maintenance cost record. These prevailing methods in maintenance accounts indicated that other procedure would be required in these cases to obtain adequate data on the conditions in service of soil-cement roads.

In order to extend the record of the condition in service of these early soil-cement road projects, this committee prepared another questionnaire designed to determine the actual square yards of soil-cement base requiring repair, the time, location and cause of failure if evident, the history of construction of the base, together with similar data with reference to the bituminous surfaces and the daily and total traffic on the roads.

The questionnaire was forwarded in 1946 and 1947 to the 23 states included in the 1940 condition survey and also to eight other states that had constructed soil-cement roads to this time, or a total of 31 states and 128 soil-cement base projects. A greater number of states and projects, were thus included in this condition survey than in 1940 or the maintenance costs of 1946 as reported in Table 1. Also, improved personnel in many Highway Departments made possible increased details in reports for this survey. Nineteen of the states were able to obtain, or had available, data to prepare and return reports covering 59 projects.

Considerable data was supplied by this questionnaire which included length, width and depth of project, the type of soil processed, cement content used, daily truck and total traffic, age of project at time of survey, age at which first failure occurred and square yards of total failed areas of soil-cement base. Additional data, not practical to tabulate for comparison and study in this report, gave the age of the base at time first surfacing was placed and the type of surfacing. Many reported the type and time of subsequent surface courses and present serviceability or condition. A very complete history of these projects from their construction up to the present, is provided by this survey.

Table 2 includes the data on soil-cement base for all projects reported by the several states. It was impractical to tabulate the many variables in bituminous surfacing.

Summarizing the data on soil cement base, the width varied from 10 to 35 feet, and depth of treatment from 5 to 10.25 inches. Forty-eight of the 59 projects reported were six inches thick.

The cement content used varied from 6 to 14 percent by volume with the majority of the work being built with 10 percent by volume. The soils processed had liquid limits varying from 2 to 72 and plasticity indexes varying from non-plastic to 35. A considerable number had plasticity indexes of the order of 10 or more.

The age of the projects at the time they were surveyed varied from 2 years to 10 years, with the majority being 5 years old or more.

Data was submitted on a total of 59 projects representing 273 miles of road. On 32 of the projects there were no failures of any nature reported for the soil-cement base. On 27 of the projects some soil-cement base failure was reported, but on 18 of these projects the failures were of a very minor extent, on three projects the breakage was less than 5 percent and was not significant. This leaves only six jobs on which the extent of breakage was sufficient to warrant analysis.

The Kern County, California project on Route 139 had 2500 square yards or 20 percent failure, with the report that considerable of the surface was dry, dusty and not knit together, indicating inadequate construction control.

The Cheboygan County, Michigan job on M-16 had 59 percent reported as replaced and this included an unspecified amount removed during subsequent changes in grade on the project. Professor Housel advised in committee meeting that more details on the reasons for replacement would be obtained and supplied to the committee.

Fifteen percent breakage was reported on the Nuckolls County, Nebraska project. Breakage was reported due to insufficient base thickness (5 inches) for the type of subgrade and traffic, low cement content for type of soil processed, together with poor construction practices.

Ten percent breakage was reported on the Thayer-Fillmore, Nebraska project. Breakage was reported as probably due to insufficient base thickness (5 inches) for the very poor subgrade soils that existed on the project.

On the Lewis County, New York project, 10 percent breakage was reported on this single lane construction, but data was not submitted to indicate the nature of breakage.

On the Erie County, Pennsylvania, Route 25018, project, 33 percent of the area was broken as a result of frost heave induced by a high water table in a silt loam subgrade.

The foregoing record of breakage on a few jobs indicates that major factors involved included faulty subgrade and improper construction. The overall record is excellent, particularly when it is compared with common experiences with low cost construction.

An interesting fact is supplied by the data on the age at time of first failure. It will be noted that with five exceptions the reported breakage occurred during the first two years of service, which indicates further that the failures were due to subgrade conditions or faulty construction. Definite data were submitted showing that breakage at later ages was due to unusually heavy trucking at time of breakage

TABLE NO. 2
CONDITION SURVEY OF SOIL-CEMENT ROADS
1946-7

Location: State	Location: County	Route	Length, Miles	Width, Feet	Depth, Inch	Cement % by Vol.	Soils Processed: PRA Group	Soils Processed: L.L.	Soils Processed: P.I.	Daily Traffic: Trucks	Daily Traffic: Total	Age of Project, Years	Age at Time of First Failure Years	Failed Sq.Yds	Total Failed Areas %
Ala.	Marshall	32	4.04	24	8	9	A-2,4	14-26	0-5	50	500	2	None	None	
Ariz.	Cochise	666	1.82	28	6	10-12	A-2	20-24	1-3	30	100	6	None	None	
Calif.	Contra Costa	106	3.35	24	5.5	8	A-2,5	20-37	4-12	235	2250	7	None	None	
"	Kern	466	1.02	24	10.25	6	A-4	38	16	47	518	8	None	None	
"	"	139	1.00	22	6	9	A-2	-	-	197	1129	8	1 (a)	2500	20
"	Siskiyou	97	3.00	22	6	9	A-2	22	0	52	777	8	None	None	
Del.	Kent	Dover-Wyoming	2.91	18	6	10	A-1	20	5	100	1000	6	None	None	
Ga.	Calhoun	62	11.63	21	6	6	A-2	9-20	-	105	339	6	None	None	
"	Coweta	14	12.35	(b)	8	10	A-2	-	-	265	1422	3	None (b)	None	
"	Clarke	24	2.0	21	6	8,10,12	-	37-58	1-3	-	362	8	None	None	
"	Fulton	14	10.42	(b)	8	9	A-2	-	-	265	1422	2	None (b)	None	
"	Liberty - Liberty Field		1.33	20	6	6	A-2	-	-	7	117	3	None	None	
"	Twiggs	127	0.86	35	6	10	A-2,4	28-44	12-21	-	288	6	None	None	
Ill.	Logan-Menard	SA3&5	2.90	20	6	14	-	35	16	90	185	7	2 (c)	90	-
"	Tazewell	461	3.86	20	6	10	-	23-24	9-8	-	500	5	1	16	-
"	Winnebago	SA26	1.14	18	6	10	-	14-22	0-5	-	50	10	7 (d)	580	5
Ky.	Caldwell	139	11.0	20	6	10	A-4-2	-	-	-	-	5	-	93	0.1
"	Daviess	144	1.0	19	6	10	A-4	-	-	-	-	4	-	105	0.9
"	Daviess	144	2.3	19	6	10	A-4	-	-	-	-	4	-	7	-
"	Daviess	144	6.54	20	6	8,10	A-4,6	20	2	180	450	7	-	34	-
"	Henderson	54	9.2	20	6	10	A-4,A-7-6	-	-	-	-	5	-	25	-
"	Magoffin	7	7.0	19	6	10	A-2,toA-7-5	-	-	-	-	3	- (e)	3085	3.9
"	Ohio	54	4.9	20	6	8,10	A-4	-	-	-	-	4	-	None	
"	Webster	56	8.9	20,24	6	10,12	A-2,4	-	-	-	-	4	- (f)	9	-
Maine	Androscoggin	136	0.67	18	6	12	A-3	-	-	45	191	6	None	None	
Mich.	Cheboygan	M-16	1.27	20	6	8-12	A-2,4,6	-	-	-	-	9	1 (g)	9400	89
"	Crawford	M-72	1.57	22	5	8-10	A-2	-	-	-	196	5	None	None	
"	Ingham	M-92	3.02	22	6	8	A-2	11-20	0	-	403	5	None	None	
Minn.	Fillmore	139	3.0	24	6	14	-	40	17	-	-	5	1	196	0.5
"	Marshall	1	6.2	24	6	10,12,14	-	22-36	0-17	-	-	4	1 (h)	2282	0.2
Miss.	Chickasaw & Pontotoc	15	23.78	22	6	10	A-2F & A-2P	24-34	0-9	-	-	8	None	None	
Neb.	Fillmore-York	81	7.35	29	6	8	-	20-24	2-6	138	690	4-5	None	None	
"	Grant	2	3.80	24	5	6,8,10	-	25	4	50	190	7	None	None	
"	Platte	81	14.77	29	6	8	-	18-25	2-4	173	680	4-5	None	None	
"	Lancaster	33	5.01	24	5	10	-	35	13	131	810	8	1	100	0.1
"	Madison-Pierce	81	4.46	24	5	8,12	-	2-34	2-12	264	1340	8	None	None	
"	Nuckolls	281	13.56	24	5	7.5,8,8.5	-	30	8	138	555	7	1 (i)	28000	15
"	Thayer-Fillmore	81	5.43	24	5	9,10,11	-	23	7	203	915	7	3 (j)	7500	10
"	York	81	7.79	28	6	8.5	-	25	4	182	990	3	None	None	
Nev.	Churchill	61	2.06	24	6	8*	A-2,3	17-23	NP	9	43	6	None	None	
N.J.	Hunterdone	Mt.Airy-Bowne Sta.	1.22	16	6	-	-	-	-	-	-	7	2	75	
"	"	(Ringoes-(Croton	1.20	18	6	10	A-2-4	26	6	25	100	8	2	60	
N.Y.	Lewis	County	1.13	10	6	10	A-2,3	-	-	-	50	7	1 (k)	800	10
Okla.	Caddo	S-8	5.8	25	6	8-10	A-2,4	28	9	266	945	7	3	91	0.1
"	Woodward	S-15	6.8	24	6	8-10	A-2,3,4	17-39	0-19	220	832	6	None	None	
Pa.	Erie	25018	3.31	16.5	6	10	A-3	-	16-22	86	692	5	0.5 (l)	9000	33
"	"	25025	1.43	14	6	10	A-3	7-23	-	46	258	5	3	1	-
"	"	25068	5.16	16.5	6	10	A-3	-	-	68	366	6	0.5	140	-
"	"	25116	3.90	14	6	10	A-3	-	-	48	276	5	0.5	20	-
"	Lebanon	38050	1.97	14	6	10	A-2,3,4	25	6	31	213	8.5	None	None	
Va.	Accomac	695	2.95	18	6	12	-	15-16	NP-3	-	*	5	None	None	
"	Gloucester	198	7.26	18	6	10,12	A-2	16-35	2-18	45	103	2	None	None	
"	New Kent	168	1.96	20	6	8,10	-	23-43	6-19	79	295	8	None	None	
"	Powhatan	60	1.00	22	6	12	-	18-25	NP-3	152	1447	5	None	None	
"	Prince Edward	134	0.50	-	6	12	A-7	65-72	22-35	-	-	2	None (m)	None	
"	Sussex	40	2.47	20	6	10,12	-	18-45	NP-16	55	335	5	3 (n)	34	0.1
"	York	168	2.80	20	6	8,10	-	20-27	4-8	195	1038	7	4 (o)	800	2.5
Wash.	Spokane	3H	3.58	20	6	10	A-4	28-31	6-11	55	275	4	None	None	
"	Whitman	3H	1.00	20	6	7.4	A-2,4	32	10	25	120	8	None	None	

NP — Non-plastic *by weight —not shown

(a)—Considerable dry, powdery soil-cement between mat and soil-cement.
(b)—Used for patching and widening.
(c)—Large coal trucks. Breakage caused by peculiar soil conditions.
(d)—Breakage due to hauling 4020 loads of crushed stone on soft subgrade in cut section.
(e)—Breakage at locations of poor drainage and bad subgrade reported as due to an unusual amount of logging truck traffic.
(f)—Information submitted showed considerable surface ravel and some settlement.
(g)—Includes soil-cement removed during subsequent changes in grade.
(h)—Failures induced by heavily loaded beet trucks traveling and parking on one side of road.
(i)—Failures primarily due to bad subgrade conditions, insufficient base thickness (5") for type of subgrade and traffic, low cement content, and poor construction practices.
(j)—Failures due to insufficient base thickness (5") for the very poor subgrade and traffic.
(k)—One traffic lane paved.
(l)—Breakage caused by frost heave of silt loam subgrade.
(m)—Closed to traffic soon after completion due to failure of adjoining section of stabilization.
(n)—Failures due to poor drainage.
(o)—All breaks on one side of road due to heavy hauling when Camp Peary was built.

and in two cases to poor drainage and bad subgrade conditions. In the fifth case relative to breakage at later ages no date was given, but the breakage amounted to only one square yard and is of no significance.

The traffic reported on the projects in this condition survey varied from a minimum of 7 trucks per day to a maximum of 265 trucks per day, with total traffic varying from a minimum of 43 vehicles to a maximum of 2250 vehicles.

There is no evident correlation between the type of soil processed, truck traffic and breakage. It is, therefore, indicated that the 6-inch depth was adequate for the variables encountered on the projects reported to date which covers many typical conditions in the several states.

All the data included in the tables on soil-cement base maintenance cost and soil-cement base condition indicate that these many roads are serving their purpose quite adequately and economically for the variable conditions of climate, subgrade and traffic encountered. A report on total mileage of soil-cement roads built to date was requested from the Portland Cement Association, which shows more than 1500 miles of soil-cement roads have been built in 39 states.

The very valuable detailed information supplied by the several states and the individuals who inspected the projects and prepared the reports, is very much appreciated by this committee and particularly from the fact we are fully aware of the personnel shortages with which they were confronted.

The value of these historical data on soil-cement roads will be greatly increased if continued studies can be evaluated after five years or more of additional service. It is, therefore, the desire of the committee that each state follow up the service behavior of all earlier projects, both for their own information and to make the data available for comparative studies and reporting in subsequent years.

The committee discussed the use of soil-cement and soil-cement mixtures for bases for concrete pavements to prevent pumping and distortion on heavy textured soils, for use as back fill at bridge abutments and to reduce volume changes and displacement problems in earth fills. The production of cement-modified soil binders in granular base materials was discussed. Their possible economy was explored for areas where use would result in lower construction costs than would result from the use of soil-cement bases involving fine grain soils, predominantely clays and high cement contents of the order of 16 percent by volume or more.

It was the sense of the Committee that these several uses of soil-cement and soil-cement mixtures warrant further study and investigation.

REPORT OF COMMITTEE ON SOIL CALCIUM CHLORIDE ROADS

H. F. CLEMMER, *Chairman*, HENRY AARON, SHREVE CLARK, L. D. HICKS, W. J. HUTCHIN, F. V. REAGEL, G. R. RICHARDSON, H. R. SMITH, J. F. TRIBBLE, E. A. WILLIS

The Committee on Soil Calcium Chloride Roads has divided its program of study into three phases.

1. The review of all available data and the preparation of a bibliography.
2. Laboratory studies through research and tests to evaluate the use of calcium chloride with various types of soils and to determine phenomena which will assist in planning field projects to determine the quantitative value of calcium chloride for stabilization.
3. The study of experimental field projects to more definitely evaluate the results of laboratory studies under actual field conditions.

The first phase of the work has been completed and a bulletin presenting a complete bibliography of available information, compiled by Dr. Cuthbert then of Princeton University, and published by the Highway Research Board in 1945 as Research Report 2-F. The bibliography has been a most important factor in the further work of the committee.

A study of the various reports and papers as called to the attention of the Committee by this bibliography and particularly those papers presented in the proceedings of the Highway Research Board during the past few years definitely showed the qualitative value of calcium chloride for stabilization of soils for roads and bases. It is, however, of importance to determine the quantitative value of calcium chloride in soils. Laboratory experiments have been conducted to establish the direction of the field research and eliminate unprofitable phases of the research. The most comprehensive laboratory research conducted by the Committee was a project conducted in the soils laboratory of the civil engineering department of the University of Maryland under the direction of Professor Morgan Johnson. Professor Johnson presented a report before the Research Board last year. Some further study has been carried on by Professor Johnson which will be included in the report of the committee but is not to be presented on this program.

A laboratory project was also carried on at Purdue University under the direction of Professor Woods and reported in the Proceedings of the Board, Vol. 26, by Mr. Yoder. This study was most extensive and presented results of wide interest. A further program of study as to the "Effect of Calcium Chloride on the Compactive Effort and Water Retention Characteristics of Soils" and a report was presented by Professor Yoder at the 27th Annual Meeting of the Board and is included in Vol. 27 of the Proceedings.

With this laboratory experience as a background the Committee believed there was ample justification for the construction of field projects this year and two such projects have been promoted--one in Virginia with the cooperation of two members of the Committee, Mr. Shreve Clark and Mr. Shelbourne and one in Alabama with the cooperation of Mr. Tribble, Construction Engineer for the State

Highway Department. These are cooperative projects between the Highway Research Board and the respective highway departments.

Mr. A. U. Theuer, formerly with the National Bureau of Standards and now Research Engineer for the Highway Research Board discusses the programs for these field studies in his paper which is included in this Bulletin.

The Committee has held one general meeting this year and several of the members have been able to visit the field projects in Virginia and Alabama and provide assistance in the work.

The Committee believes it would be extremely desirable if a series of such experiments could be conducted on a large scale among the states making use of granular stabilization. Such tests should be correlated and should cover as wide a range of soil types as possible. Accordingly, the Committee is planning for other field projects for the coming season. It is believed the results of these studies will provide data upon which a more rational approach to the design of stabilized roads may be made.

FIELD STUDIES TO DETERMINE THE VALUE OF CALCIUM CHLORIDE FOR COMPACTION OF SOILS

A. U. THEUER, *Research Engineer*
Highway Research Board

The favorable results attained in the laboratory with the use of calcium chloride as an integral admixture in sand-clay base materials, has led the Soil Calcium Chloride Roads Stabilization Committee, to extend the scope of its work to full scale field investigations.

During the past summer the Committee initiated two cooperative projects and formulated plans for several more next year. Due to delays in getting the actual work underway, however, construction on only one of two projects has been completed. This is located in nearby Virginia. The second project, located in Alabama was started last month and is now in progress.

As stated by the Committee, the purpose of these field investigations is to study construction methods, durability, and performance, as well as those primary factors, density, moisture, compactive effort and strength. Each project following construction is to remain under observation for a minimum period of one year. Time will also be spent in a comparative study of methods for determining strength relationships.

I will give a very brief review of the Virginia project, and a few data just by way of indicating what is being undertaken.

Several possible locations for starting the first investigation were made available by the Virginia Department of Highways last August. From these, a 3½-mile reconstruction project located in the Coastal region was selected. The reconstruction called for a 10-inch stabilized sand-clay base and a two coat asphalt wearing surface. Material for the base was secured from a nearby pit and was placed in two 5-inch courses.

For purpose of the experiment, a 3000-foot section, subdivided into 1000-foot lengths was selected. For the first section a 2½-pound per sq. yd. treatment was used. The second or control section was untreated. The third section was given a 5 lb. per sq. yd. treatment.

The base material as received from the pit was a quite uniform sand-clay coming under the PRA classification as an A-2 material. It had a liquid limit of 21.8, a PI of 4.7, and a Standard Proctor Density of 128 lb. per sq. ft. at an optimum moisture of 9.2 percent.

The experimental sections were first brought up to approximate grade and profile. The designated amount of calcium chloride was then spread uniformly over the two treated sections. It was thoroughly mixed with the top 5 inches, compacted depth, by means of a scarifier and a Seaman Pulvi-Mixer. Water to bring the material up to near optimum was added in the course of the mixing operation. The untreated section was constructed in identically the same way as the other two, except that no chloride was added in the mixing operation.

Compaction followed mixing. Immediately after compaction the road was thrown open to traffic. Eighteen days after construction the asphalt wearing surface was placed. Maintenance during this intervening period consisted of a light blading and the addition of water to the untreated section for laying dust.

It was intended to make a series of density measurements at intervals during the process of compaction. With the equipment available, this did not prove practical. (However, measurements, to establish a relation between compactive effort and density, are being made on the Alabama project now in progress.) Compaction was accomplished by means of a self-powered sheepsfoot roller and a rubber-tired power tractor. When the first density measurements were made they were found to be approximately 100 percent standard Proctor density. No additional compaction was attempted. Just as an indication of relative densities of three sections immediately after compaction, the following values may be given:

For the 2½-lb. treatment	130 lb. per cu. ft.
For the 5-lb. treatment	125½ lb. per cu. ft.
For the untreated section	128 lb. per cu. ft. same as standard Proctor density.

Density measurements were made at 1, 5, 7 and 15 days, following construction and are now being made every 30 days.

Other measurements made before surface treatment, included pH-values of the base materials, calcium chloride contents of roadway samples, surface roughness and strength measurements.

For determining strengths, the Burggraf Shear Apparatus was adopted. This apparatus is described in detail in the 1938 Proceedings of the Highway Research Board. The apparatus consists of a calibrated jack and plunger with attached pressure gauge. A strength measurement is made by placing the jack in a carefully prepared hole dug in the roadway, and applying a horizontal thrust through a parabolic plate bearing against a vertical surface. Measurements with this apparatus are being made at different depths down to and including the subgrade.

Again, merely as an indication of relative shear strengths obtained to date the following values were found:

at 4-days after compaction:

for 2½-lb. treatment	31.5 psi.	at 4.7%	moisture
for 5-lb. treatment	17.7 psi.	at 5.3%	"
for untreated	18.1 psi.	at 4.6%	"

at 11-days after compaction these values were:

for 2½-lb. treatment	31.6 psi.	at 4.8%	moisture
for 5-lb. treatment	15.8 psi.	at 6.7%	"
for untreated	11.6 psi.	at 6.4%	"

These values are all for the top 5-inch layer.

Since the tests and test data are still incomplete, nothing in the way of conclusions can be presented at this time.

JACKED-IN-PLACE PIPE DRAINAGE

JACOB FELD, *Consulting Engineer, New York, N. Y.*

SYNOPSIS

Control of internal soil stability is chiefly a matter of moisture control. Where the desired drainage is to be installed at a considerable depth below the top of an embankment or where disturbance of a developed surface is not desirable, the economy of introducing drain pipes by static or dynamic jacking should be considered. Jacking of pipes under railroad tracks and under heavy traffic highway pavements has become standard practice. The procedures have developed as an art with little or no attempt to compute expected loads or required installation forces. This report summarizes the practical development of installation methods and evaluates empirical and theoretical resistances which may be used as a guide to the design of jacked-in-place drainage and other pipe.

Pipes may be installed closed end when sufficiently large jacking pressures can be provided and when the soil displacement will not cause undesirable heave at the surface of the fills. Small size pipes, probably not exceeding 4 inches in diameter can be inserted by the vibratory impulses from an air driven jack-hammer. To introduce larger pipe into the soil without removal of the displaced soil requires too high static pressures and has no advantages. Up to about 36 inch diameters, open end pipes must be cleaned of displaced soil by either water jet, preferably combined with compressed air, or by augers, although 24 inch pipe has been installed with hand excavation. Larger pipes can be excavated by hand, with cable drag scoops or small wheelbarrows to remove the spoil. Static pressures are applied with jacks, both screw and pneumatic types are used, modified to provide long movement in each throw to reduce the time loss in re-blocking. The forward end of the pipe is usually reinforced by a cutting edge, and in soft ground is protected by a metal shield overhanging the pipe end. Proper back-pressure resistance must be provided to take the maximum jack pressure, in the form of anchored or braced sheeting in the approach pit, or by deeply inserted piles.

SMALL PIPE INSTALLATIONS

Boring. The simplest method for small pipes is to provide a horizontally bored hole by air driven earth borer. The Hydrauger Corp. of San Francisco lists three models of boring machines which are reported to be consistently straight boring for the underground distances up to 200 ft., at rates from 20 to 40 ft. per hour. Power is provided by air driven motors, using 90 lb. air pressure, and water under pressure clears the hole. Boring bar sections are 2 ft. in length, boring tools are 2 in. diameter and reamers up to 14½ in. diameter are available. Except in liquid soils there is little difficulty in preparing a hole for the insertion of a pipe. For such small diameters, the hole will not close up and there is no danger of excessive pressures on the pipe, so that minimum standard wall thickness, consistent with expected loss from corrosion, chemical and electrolytic, can be used (Fig. 1).

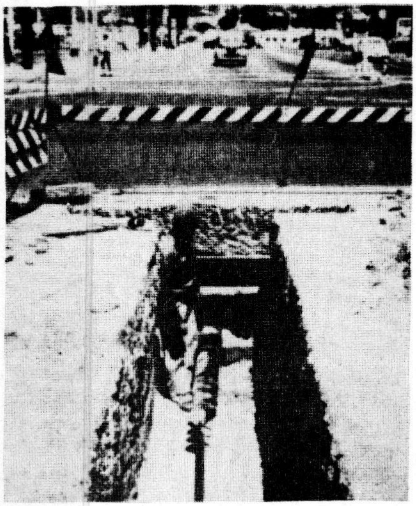

Figure 1. No Traffic Interruption

Vibrating Hammer. Closed end pipe, up to 2-in. diameter, can be inserted in natural or fill ground not containing boulders or rock fill with a standard air driven pavement breaker, using a special driving tool. The length which can be inserted is independent of the depth of cover since the radial pressure on the pipe is small. Careful alignment of hammer and pipe is necessary to prevent the pipe point travelling off line, especially when the amount of cover is small. The Atlantic Steel Co. of New York manufactures driving points and heads to fit from one to two inch pipe sizes, and driving shanks of 1-1/8 and 1-1/4 hexagon sizes to fit standard pavement breakers. The points are truncated cones with a base projecting slightly beyond the pipe. A universal joint is provided between the shank and the driving head to widen the driving range in tight working spaces. Driving speed of 25 ft. per hour in ordinary soils is claimed by manufacturers.

Jacking. Four 6-in. pipes were jacked through a railroad fill at a depth of 19 ft. in 1928, near Birmingham, Ala. Pipe was standard weight wrought iron, equipped with a forged steel point, 2 feet long, welded to a length of extra heavy pipe. Each pipe was pushed under 10 railroad tracks, for a distance of 200 ft. in 15 hours time. The jack was a hydraulic cylinder with 6 in. piston and 5 ft. 3 in. stroke, equipped with a motor driven hydraulic pump. Maximum pump pressure was 4500 lbs. per sq. in. but the usual pressure was 2000 lbs. To maintain proper alignment, about 15 ft. of pipe was held in the open approach trench between steel channel guides. (*1*)[1]

In San Diego, California, 100 ft. of gas main was laid in 1947 across a busy intersection by drilling through the sections of pipe with an auger drill, taking out 3 ft. bites of earth and jacking the pipe into the hole (*2*).

Fusion Piercing. In extremely hard ground, the process of "Fusion Piercing", patented by the Linde Air Products Co. can be employed to penetrate solid strata. The method was developed for reducing the cost and time of drilling blast holes in iron ore. A hole is made by burning a mixture of oxygen and a flux bearing fuel and directing the flame against the end of the cavity. The flame temperature of 4000 F. causes some rocks to spall and flake and others to melt. Pressure of the flame gases forces the flakes or molten material past a water spray. The sudden heat exchange further pulverizes the hot rock or soil and at the same time changes the water into steam under pres-

[1]**Italicized figures in parentheses refer to list of references at the end of the paper.**

sure within the restricted volume of the excavated hole. Such steam under pressure automatically cleans the hole of rock and soil chips. Six inch diameter holes 30 ft. deep were fusion pierced in a low grade abrasive iron ore at an average rate of 10 ft. per hr. In this type of material ordinary drilling speeds average 1 ft. per hr. Pipes inserted in holes so prepared have no exterior loads to carry, since the fusion of the annular volume around the hole solidifies the material to form, in effect, a structural tunnel which can transfer overburden and lateral loads into the undisturbed soil regions.

Water Jets Jacked. Horizontally driving of well pipes for water collection, a method developed by Leo Ranney of Ontario, Canada, can be used also as drainage pipe installations. At Canton, Ohio, 36 separate 8-in. pipes were jetted outward through the sides of a shaft, 12.5 ft. in diameter and 147.5 ft. deep. The maximum length of 8-in. screen pipe was 175 ft. Each slotted screen pipe was fitted with a hollow cast steel conical digging point having slots in the wall and connected to an interior 2-in. bleeder pipe. As the pipe was projected by hydraulic jacks, the displaced fine soil came through the screen pipe into the shaft; about 3 cu. ft. of sand was removed for each foot of pipe projected from the shaft (*3*).

A similar system was used to dewater the ground in the vicinity of the Brooklyn end of the Brooklyn-Battery vehicular tunnel in New York City. From the bottom of a vertical shaft, 11.5 ft. inside diameter and 70 feet deep, twelve 8 in. screen pipes, totalling 1100 lin. ft. were projected by hydraulic jacks. The digging head was bullet shaped with slots, connected to a 2 in. bleeder pipe to remove the displaced soil, flowing through the slots under hydrostatic pressure. The water table level was about 50 ft. above the pipes. To avoid excessive soil removal, fine screens filtered out all coarse particles which were displaced. The longest pipe projected was almost 200 ft. from the shaft (*4*).

Loads and Stresses. Where holes are prepared for the insertion of a pipe or conduit, whether by auger or fusion methods, no loads are immediately imposed on the pipe. With time, it is possible to have an internal soil readjustment which, from a vertical slip or a lateral squeeze, will impose pressure against the pipe. Since the methods are only applicable to small sized pipes, and the strength under the three edge bearing method of ASTM C-14, which probably duplicates the worst possible load application, is 1100 lb. per lin. ft. for 4- and 6-in. plain concrete pipe, no special investigation is necessary for loading stresses. The stresses of pipes installed by jetting or jacking are similar to those on large pipes, discussed in Section B6 below.

LARGE PIPE INSTALLATIONS

The method of jacking pipes under railroad fills to eliminate the disturbance to track and traffic from open cut trenching was started by the Northern Pacific R.R. prior to 1900. At first this method was limited to cast iron pipe, and soon became standard practice for several railroads. Corrugated iron pipe was first jacked in 1922; precast concrete pipe was first used in jacking operations in 1927. The smallest practical size is 30 in. and the largest size reported is a 96-in. reinforced concrete section used for a pedestrian underpass, and a 96-in.-8 ga. corr. iron culvert. There is a gap in size from 6- to

30-in., because excavation by methods other than hand shovel and and wheel barrow require special equipment. There is no justification for avoiding the sizes smaller than 30-in. because water jet excavation within a pipe can be performed under control without danger of slides and soil failures at the front of the pipe.

Concrete pipe installations are described at length in "Concrete Pipe Lines" 1942, by the American Concrete Pipe Association of Chicago. Some installations of concrete pipe by jacking methods with pertinent data are listed in Table 1.

TABLE 1

CONCRETE PIPE INSTALLATIONS BY JACKING METHOD

Item	Date	Pipe Size in.	Soil Cover ft.	Size of Jacks T.	Lengths Jacked ft.	Cutting Edge	Location
1.	1927	60	RR.	-	80	steel	Coraopolis, Pa.
2.	1930	72	12 & RR.	2-350	105	steel	Youngwood, Pa.
3.	1930	48	4 & RR.	2-screw	48	none	Rochester, N.Y.
4.	1930	42	16 & RR.	2-30 s.	76	none	Belleville, N.J.
5.	1931	30	RR.	2-250 h.	100	none	Quantico, Va.
6.	1931	36	RR.	2-250 h.	100	none	Quantico, Va.
7.	1931	60	RR.	2-250 h.	100	none	Quantico, Va.
8.	1931	66	RR.	2-250 h.	100	none	Quantico, Va.
9.	1931	60	45 & RR.	2-50	96	none	Stockholm, S.D.
10.	1932	84	3 & RR.	-	48	-	Hagerstown, Md.
11.	1932	36	8 & RR.	2-175 h.	40(stopped)	-	King Co., Wash.
12.	1935	60	4 & RR.	2-100 s.	52-32	steel	Rutland, Vt.
13.	1935	60	4 & RR.	2-100 s.	46(stopped)	steel	Rutland, Vt.
14.	1935	36	Street car	50 h.	-	steel	Buffalo, N.Y.
15.	1935	60	30 rock fill	2-100 s.	100	-	Winston-Salem, N.C.
16.	1936	60	slag dump	-	96	-	Rutland, Vt.
17.	1936	42	RR.	-	64	-	Rutland, Vt.
18.	1936	42	Highway	-	72	-	Rutland, Vt.
19.	1936	36	RR.	-	41 & 42	-	Rutland, Vt.
20.	1936	33	RR.	-	88	-	Rutland, Vt.
21.	1938	72	18 & RR.	-	96	none	Huff, N.D.
22.	1939	42	RR.	2-screw	75	-	Henrico Co., Va.
23.	1939	48	8 & RR.	2-75 s.	55	steel	Hammond, Ind.
24.	1939	48	RR.	2-100 air	70	steel	Hammond, Ind.
25.	1939	48	RR.	2-100 air	85	steel	Hammond, Ind.
26.	1939	48	RR.	2-100 air	60	steel	Hammond, Ind.
27.	1939	60	RR.	2-100 s. 4 @	100	steel	Charlotte, N.C.
28.	1939	60	RR.	2-100 s.	168	steel	Greenville, S.C.
29.	1940	96	15 & RR.	4-100 h.	69	steel	Elmira, N.Y.
30.	1942	54	14 & RR.	2-100 h.	148	steel	Warren, O.
31.	1946	48	5 & RR.	265 h.	75	steel	Chicago, Ill.
32.	1934	54		2-100	79	steel	Elwood, Pa.

Note: All of the above installations are described in "Concrete Pipe Lines", 1942, published by American Concrete Pipe Association, Chapter IV, p. 59-84, except items 11, refer. (5); 30, refer. (6); 31, refer. (7); and 32, refer. (18). Under the Size of Jacks, s. denotes screw and h. denotes hydraulic jacks.

Precast pipe used in jacking installations should be reinforced concrete culvert pipe (A.S.T.M. C76), preferably extra strength. The difference in cost between "extra-strength" and "standard strength" is small insurance for the expensive correction should a pipe section break from unequal jacking strains. Pipe lengths should be 4 ft. and where specially manufactured for the job, longer sections will expedite the work especially if the jacks have a run of more than 4 ft. Since the projections of the bells would greatly increase the soil resistance, pipe joints must not be of greater outside diameter than the pipe being jacked, even though the cutting edge may project beyond the pipe circumference.

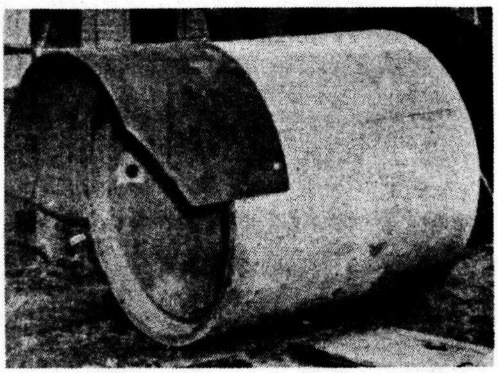

Figure 2. Part Shield

Steel pipe installations are all of corrugated or fluted plate shells, coupling short lengths together. Standard weight steel pipe can be used if the short lengths are either welded together, or for smaller sizes, connected by means of internal sleeves, such as are used for splicing steel shell piles. Some examples of steel pipe installations are listed in Table 2. Standard type Armco corrugated pipes were used, with flat slip plates sometimes covering the end

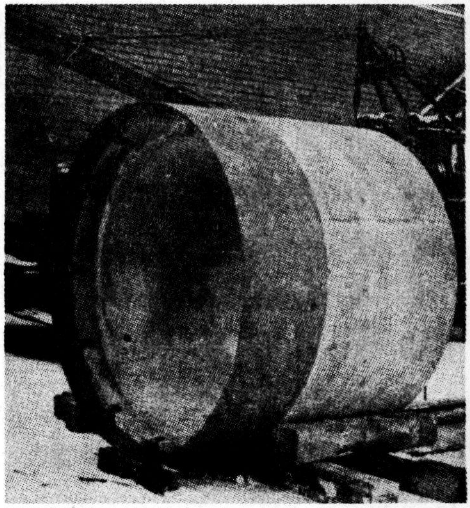

Figure 3. Full Ring Shield

of the leading section. In Item 7, the slip plates were corrugated sheets with ridges running parallel to the pipe axis, but a flat plate could have been used with the same result. In Item 1, the soil was a fine dry sand overlying hard yellow clay, both materials encountered in the heading. The fine sand ran into the corrugations and the joints between sections, causing the pipe to freeze at 26 ft. penetration. Operations were continued from the opposite end and the 100 Ton jacking pressure was sufficient to drive the 54 ft. needed for closure, where special precautions were taken to avoid sand runs into the pipe. Corrugated iron pipe is obtained in sections from 10 to 20 ft. long and provision must be made in the access pit for connection of sections by rivetting or welding to form a continuous compression unit.

Cast iron pipe was used in the earliest jacking installations, by the Northern Pacific R.R. prior to 1900, the Great Western R.R. in

TABLE 2

CORRUGATED IRON PIPE INSTALLATIONS BY JACKING METHOD

Item	Date	Pipe Size in.	Cover ft.	Size of Jacks T.	Lengths Jacked ft.	Location	Reference
1.	1927	60	-	2-50	26, 54	Dover, N. H.	(16)
2.	1929	60	7	2-25 track	80	Lorain, O.	(17)
3.	1930	3-36	5	screw	14, 15, 20	Crosby, Minn.	(8)
4.	1931	52	9	screw	32	Nashville, Tenn.	(15)
5.	1932	36	8	1-200 h.	118	King Co., Wash.	(5)
6.	1936	42	-	-	30-50-(buckled)	Vermont	(9)
7.	1937	36	66	2-50 h.	210	Pittsburgh, Pa.	(10)
8.	1927	42	5	2-50 s.	4 @ 70	Buffalo, N. Y.	(19)(24)
9.	1927	12	15	1-35 h.	36	Imperial Co., Calif.	(20)
10.	1929	48	34	2-50 h.	126	San Diego, Calif.	(21)
11.	1926	60	30	2-50 s.	68	Plattsmouth, Nebr.	(22)
12.	1926	42	47	2-50 s.	80	Varna, N. Y.	(23)
				1-50 s.	25		
13.	1926	42	-	-	56	Claremore, Okla.	(22)
14.	1926	36	11	1-25 s.	40	Exeter, N. H.	(25)
15.	1927	36	-	1-50 s.	35	Sheridan Co., Kans.	(26)

See Table 1 for notation. References to installations are given under numbers in parentheses. Armco Drainage & Metal Products, Inc. report completion of 830 separate pipes totalling about 56,000 lin. ft., in the period 1922-1947, of corrugated metal pipe from 28 to 96 inches in size.

1911 and Southern Pacific R.R. in California in 1915. Credit is given to Mr. Augustus Griffin, now of Calgary, Alberta, for popularizing the jacking method of pipe installation below railroad tracks. Concrete and corrugated steel pipe have entirely replaced the use of cast iron because of the smaller approach pits necessary and the easier manipulation of the shorter lengths of pipe.

Excavation Methods. Pipes have been jacked in all types of soil, from liquid mud to rock fills. Excavation procedures are dependent upon the soils encountered. Generally drainage of ground water can be automatically provided by starting the pipe from the lower end and pumping from a sump in the access

Figure 4. Excavation at Heading

pit. In very wet areas, well points are used for lowering the ground water and may even be inserted in the heading for stiffening up running soil.

Hand excavation methods are limited to pipe installations of 30 in. and greater sizes. Short handled picks and shovels, not more than 18-in. long can be used to loosen hard soil. In tough clay soils, pneumatic spades cut the heading to proper shape. Boulders can be broken by air hammers or by applying heat and water. Oil torches have been used for this purpose.

The size of the excavation depends upon the soil type and the possible damage from loss of soil. In hard ground, excavations can be as tunnels 4 ft. or more ahead of the lead pipe and several inches larger in diameter, so that the jacking pressures required are low. To prevent the pipe from moving out of line or level, the excavation should be about one inch outside of the pipe at the top and sides, but the bottom must be cut neat. In soft ground, no cutting ahead of the lead pipe is possible, and in very large pipe jobs, timber bulkheads may be needed at the face to brace the heading and prevent cave-ins. In the 8 ft. installation (Item 29, Table 1), a louvre bulkhead was provided, permitting excavation at four different levels.

Excavation in wet and loose soils is difficult and de-watering methods are advisable. Where the ground is soft, or where, as under railroad tracks no loss or loosening of the soil cover can be permitted, a cutting shield should be extended from the lead pipe. The usual shield is a 3/8-in. steel sheet bent to the shape of the pipe and fastened by bolts or otherwise to the outside of the shell. Projection of the shield from 6 to 18 inches has been used in various installations. With a shield, the excavation must not encroach within the normal slope of the exposed soil, with the cutting edge of the shield fully buried. The shield need not extend lower than the mid-height of the pipe.

In plastic clays, even though there are no cave-ins, loss of ground from the expansion of the exposed heading may cause settlement of the supported track or roadway. In such soils a cutting edge is advisable and excavation should not be carried to the end of the lead pipe. Experience indicates that 18 inch unexcavated protection (for a 60 in. pipe) is sufficient to prevent loss of ground.

Pipe lines will not maintain proper alignment as excavation and jacking advance the heading unless proper precautions are taken. In granular soils, if the joints are not kept free of sand grains, the pipe will rise. In clay soils, a shield will keep the pipe in line. In the example listed as Item 2, Table 1, a steel bar 4 feet long was attached to the crown of the lead pipe to act as a guide into the soil and prevented deflection of the pipe from the desired line.

The approach pit excavation must be large enough to enclose a full length of pipe section, the jacks and their back-stops, together with working room for operation of the jacks and for connecting the pipe sections. The subgrade of the pit should provide a guide rail set-up on which the pipe section will rest and slide in the proper direction and desired slope (Fig. 5).

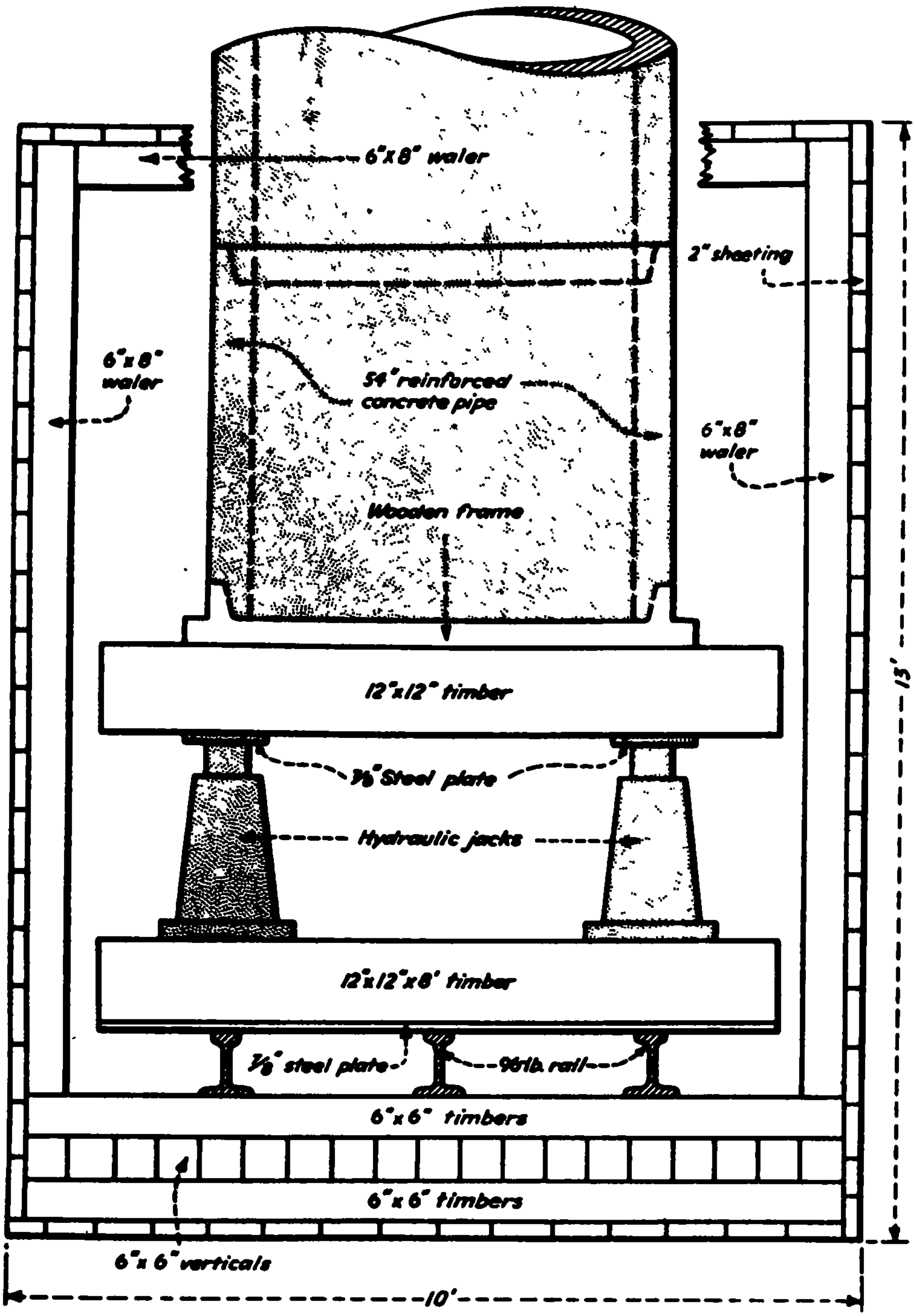

Figure 5. Typical Access Pit Set-up

Progress of pipe installation is dependent upon speed of excavation. In the Pittsburgh job (Item 7, Table 2) a 36-in. corrugated iron pipe progressed about 10 ft. per 3 shift day average during the 200 ft. of jacking. Similar progress rate is reported in the 36-in. corrugated iron pipe line 118 ft. long installed in King Co. Washington, in water-bearing clay and fine sand strata (Item 5, Table 2). In

Lorain, Ohio, a 60-in. Armco pipe in very compact clay was advanced 1.22 ft. per hour (Item 2, Table 2). In Hammond, Ind., work (Items 24, 25 and 26, Table 1) 48-in. concrete pipe was advanced from 14 to 17 ft. per 8 hr. day in water bearing sandy soil, with water table lowered by well-points, and no excavation allowed ahead of the lead pipe. Daily progress in completing 60 and 72-in. concrete pipe under railroad tracks (Items 21, 27 and 28, Table 1) varied from 5 to 7 ft. although the earlier recorded installations seemed to tend to continuous operation, to avoid "freezing of the pipe", the time interval between single shifts per day is not sufficient to allow an appreciable consolidation of the disturbed ground at the pipe surface, and no trouble has been found from the more economical single shift per day work schedule.

Jacking devices employed are screw jacks, ratchet jacks, air piston cylinders and hydraulic jacks. Only in a well made hydraulic jack can any record be made of acting pressures. The greater ease of operation and better control make it advisable to use hydraulic jacks exclusively.

The operation is an intermittent pushing of the pipe forward to the full stroke of the jack piston, retraction of the piston and insertion of blocking, then another push forward. The longer the jack stroke, the fewer stops in forward motion. The ideal set-up is to have a jack stroke somewhat longer than the pipe sections, so that no intermediate temporary blocking is required (Figs. 6 and 7).

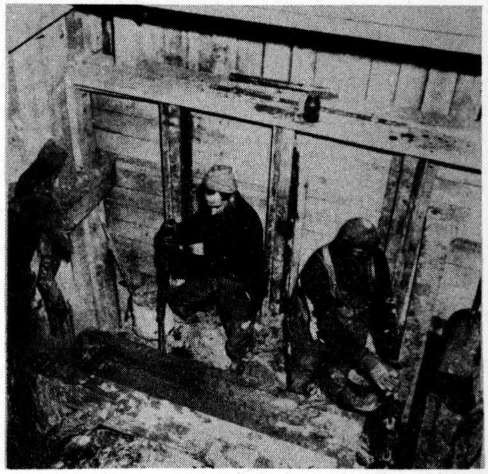

Figure 7. Access Pit Arrangement

Note: Fig. 6 from Armco Culvert Mfts. Co.
Figs. 1-2-3-4-5-7 from Eng. News-Record.

Correction of alignment can be accomplished by eccentric pressure application, with single jacks. This is done by offsetting the con-

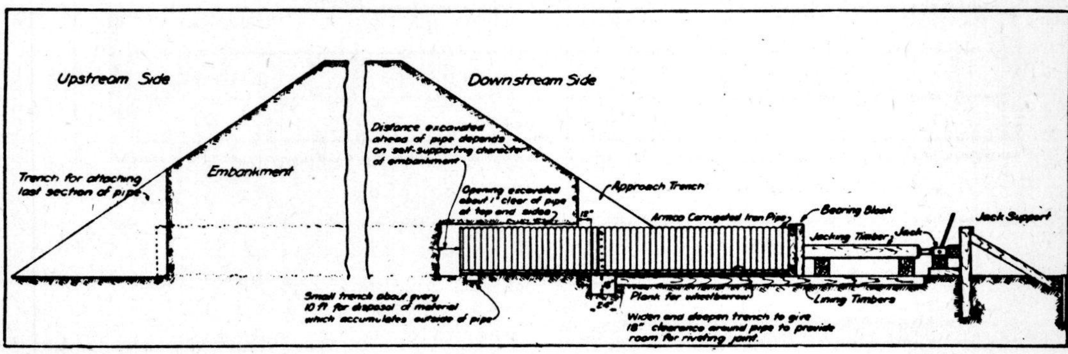

Figure 6. Diagram of Operation for "Armco" Pipe

tact between jack and jacking frame. With multiple jacks, the eccentricity can be applied by varying individual jack pressures, without changing the set-up of equipment. Another advantage of multiple jacks is the possibility of "rocking" the pipe to reduce surface friction, by working the jacks separately for short periods (*11*).

The frame distributing jack pressure into the pipe should be designed to transmit the maximum jack force into the pipe without too much deflection, which would mean a loss in jack extension, and with fairly uniform load application on the full perimeter of the pipe. A wood ring frame to fit the groove of the pipe section, with grillage beams for taking the jack concentrations is found most suitable.

Sufficient passive soil resistance must be developed for the reaction of the jacks. This is usually accomplished by tight sheeting at the back of the access pit, with vertical beam or rail framework, against which the jacking support rests. Where no access pit is required, a backstop of timbers, often railroad ties is set into the ground, with a diagonal support for the top reacting against imbedded anchorages. The amount of resistance to be used should be half of the maximum passive soil resistance on the imbedded sheeting area, to avoid excessive deflections and movements into the soil. Resistance of various types of soil and the resisting value of imbedded anchors and piles are summarized in Proceedings Highway Research Board, 1943, p. 403 (*12*).

An unusual type of resistance was devised at the Quantico, Va. installation of four concrete pipes running through a railroad fill located in tidal swamp lands (Items 5, 6, 7 and 8, Table 1). Four 2-in. iron pipes were first pushed through the fills at each installation, so located that they are outside of the pipe diameter. Cables were threaded through the pipes, anchored to a timber bulkhead at the far end and tied to a timber backstop against which 2-250 Ton hydraulic jacks reacted.

Loads and Stresses which must be considered in a jacking installation depend on the type of soil and the method of operation. The earth load carried by a pipe during installation is probably less than the load after the disturbed soil volume has consolidated. The amount of jacking pressure required to overcome frictional resistance is also dependent upon the type of soil and the method of operation, as well as the length of pipe imbedded. There are a number of cases which must be separately considered.

Case A. Open end pipe jacked through a bore excavated ahead of the pipe.
Case B. Open end pipe jacked into the soil with excavation inside the pipe only.
Case C. Closed end pipe jacked through the soil without any excavation.

Little empirical data is available to check theoretical analysis of any of these cases. Where the amount of jacking pressure at any stated conditions is known, the total radial earth pressure on the pipe can be computed, if the weight of the pipe and the coefficient of friction between pipe and soil, and the coefficient of internal friction of the soil have been recorded. Some idea of the range of probable values can also be obtained from the pulling resistance of piles and from the recorded data on shield tunnel jacking pressures.

The penetration of a pipe laterally into the ground is a somewhat different case than the introduction of a pipe vertically into the ground. The frictional resistances, however, are similar, even though in the former case the internal stress of the soil penetrated, prior to the disturbance, is fairly constant, because both are problems in viscous flow. In viscous or linear flow, displacements result from shearing forces acting over a time interval. The frictional resistance of the soil is then

(1) independent of the external loads on the surface of failure.
(2) equal to zero if the velocity is zero and is directly proportional to the velocity of relative displacements along the surface of failure.
(3) proportional to the area of the surfaces in contact.

The determination of bending stresses in the pipe during installation is not necessary, since the loadings are much more uniform than under the worst possible static loading after the surrounding soil has consolidated around the complete pipe. The pipe strength as a column must be investigated, although experience shows that the adjacent soil, even if the pipe is being pushed into a pre-excavated tunnel, provides sufficient lateral support to prevent buckling.

Case-A. The amount of jacking pressure required for any set of conditions can only be approximated. In Case A, the only resistance is along the outside of the imbedded pipe and that can act only where the soil is in contact with the pipe. Where the ground penetrated has sufficient internal shear resistance to permit the excavation of an unlined tunnel without tension failure and falling down of the overhanging material above the tunnel and without lateral squeeze at the sides, the only starting resistance to be overcome is the frictional component of the pipe weight resting on the bottom of the excavation. Krynine in 1945 analyzed the internal soil conditions which are necessary for such behaviour (*13*). Light weight corrugated iron pipe requires less jacking force than concrete pipe, where the soil can be put into a self-sustaining tunnel, because of smaller contact surface with the soil. This excavation procedure must not be allowed where any lateral soil movement to fill the voids along the perimeter will cause settlement of track or pavement at the top of the fill, and, of course, is especially dangerous in shallow fills.

In four examples of Case A installations, there are enough recorded data to evaluate the surface resistance:

(1) Item 7, Table 2, a 36-in. corrugated pipe penetrated 210 ft. in a compacted earth fill 66 ft. deep, with only 100 Ton jack pressure available. If all the pressure was used, and the entire perimeter were in contact with the soil, the resistance was 101 lbs. per sq. ft. of pipe surface.
(2) Item 5, Table 2, a 36-in. corrugated metal pipe penetrated 117 ft. in wet clay and mostly running sand, with a 8 ft. cover under railroad tracks. During the first 60 ft. jacking at the rate of less than one foot per hour, pressure increased uniformly to a 20 Ton Value, or 50 lb. per sq. ft. of the exterior pipe surface. Rate of progress for the rest of the job was increased to 1-1/2 ft. per hour. The jacking pressure still increased uniformly with distance, but at the rate of 0.5 tons per ft. penetration, or 106 lb. per sq. ft.

resistance.

(3) Item 30, Table 1, a 54-in. concrete pipe with 5-1/2-in. wall thickness penetrated 148 ft. in a dry clay soil with a large percentage of gravel, 14 ft. deep, with an actual maximum jack pressure of 1,60 Tons. If the entire pipe perimeter were in contact with the soil, the resistance was 127 lb. per sq. ft. of pipe surface.

(4) Item 9, Table 1, a 72-in. concrete pipe with 7-in. wall thickness penetrated 96 ft. in compacted dry black loam and clay, a maximum of 45 ft. cover, with only 100 Ton jack pressure available. If all the jack pressure was used, and the entire pipe perimeter was in contact with the soil, the resistance was 92 lb. per sq. ft. of pipe surface.

In installation of Case A type, it would seem to be sufficient to expect a jack pressure requirement of 100 lb. per sq. ft. of total soil imbedment of the pipe perimeter, with provisions for about 25 percent overload. The actual resistance per sq. ft. of contact is probably three times this value, but contact over more than 1/3 of the surface is not possible in Case A.

After completion, the soil will eventually come into intimate contact with the pipe over its entire length. In the type of soils where Case-A method can be used, the probability of load first coming on the roof of the pipe is great. This will be equivalent to the loading known as "Minnesota Bearing" in the laboratory tests of pipes. Spangler (*14*) evaluates this loading as 1.37 times the severity of the sand bearing test and 0.91 as severe as the three-edge bearing test. The comparable pipe bedding condition for the most favorable result after soil consolidation above the pipe is what Spangler calls "First Class", ditch bedding, (*14*), but there is the possibility of the more severe loading, especially if rigid pipe is used. A maximum load, lb. per ft. of pipe, of 400 d^2, where d ft. is the outside diameter, seems to be a reasonable value for the necessary strength of the pipe.

Case-B. In Case B, in addition to the friction along the outside of the imbedded pipe length, there is the force necessary to squeeze out the soil immediately in front of the wall thickness of the lead pipe. A shield or cutting edge reduces the letter resistance by separating the soil to be placed into (1) an annular ring under compression, and (2) interior soil in motion towards the excavation bounded on the outside by the steel cutting plate. The stress conditions in this case, after completion, are substantially identical to those in a shield driven tunnel, where no compressed air is used and the excavation is 100 percent of the tunnel section. During installation, the jacking pressure for a shield from 16 to 20 long advanced 20 to 30 inches is not comparable with the case of a pipe, 100 ft. or more in length, being moved bodily. The soil immediately adjacent to the pipe circumference is in viscous motion, during the jacking operation, and the shear resistance of the soil is independent of the depth of the cover, and is a direct function of the area of imbedded pipe and of the velocity of propulsion. The effect of vibration, from traffic at the surface of the fill, is to reduce the viscosity of the soil in motion, and therefore reduce the necessary jacking pressure. This result was noted in several of the installation records. Another favorable factor often mentioned is the surprisingly small load required to start the pipe moving again, at the beginning of the shift, and the lack of "freezing" over night. The explanation is, of course, the low velocity of motion at the resumption of work.

In two examples of Class B type, the actual jacking pressures required were measured and recorded:

(1) Item 29, Table 1, a 96-in. concrete pipe with 9-in. wall thickness penetrated 69 ft. with jacking pressures increasing almost exactly with length of imbedment at the rate of 4.5 tons per lin. ft., in a miscellaneous fill 6 ft. deep and under a series of railroad tracks. The resistance at the pipe surface in contact with the soil was 300 lb. per sq. ft.
(2) Item 31, Table 1, a 48-in. concrete pipe with a 5-in. wall thickness penetrated 75 ft. in fine sand and some gravel, drained by well points, with 5.5 ft. cover under a series of railroad tracks. At 55 ft. imbedment, the jacking pressure was 127 tons, or 300 lb. per sq. ft.; at 75 ft. the jacking pressure was 185 tons, or 325 lb. per sq. ft. Necessary pressure to maintain forward motion decreased as trains passed over the site, and the pressure necessary to start motion after a period of stoppage, was less than that required to maintain constant forward movement.

The jacking pressure capacity for a shield driven tunnel, where forward shield motion is about 2 inches per minute, is from 3 to 4 tons per sq. ft. of shield perimeter surface, although actual pressures required are only a part of the capacity. The problem differs from that of pipe jacking because of the tunnel face resistance developed by the rigid shield pressing against the outside soil, and the stiffening up of the subaqueous silt, sand or clay by the constant leaching of the compressed air through the face of the shield. Rate of progress during jacking is at least ten times as fast as the normal pipe operations.

Jacking pressure for a Case-B pipe installation should be provided to allow from 300 to 350 lb. per sq. ft. of imbedded surface. This value is substantially equal to the ultimate resistance of average type clay and silt soils.

After completion, the pressure on the imbedded pipe, with the outside surface in intimate contact with the soil on the entire perimeter, is less than in the most favorable ditch condition analysed theoretically and experimentally by Spangler, (*14*), so that the standard reinforced concrete or steel drainage pipes can be safely used.

Case-C. Pipes introduced with closed ends by either static or dynamic loads are similar to piles being jacked or driven horizontally. Considerable resistance to penetration can be eliminated with water jets, although normal conditions will not require such aid. The closed end pipe must be pushed by a force larger than the frictional resistance acting on the surface of the entire length of imbedded pipe plus the force necessary to squeeze the displaced soil into the adjacent volumes. The frictional resistance to overcome can be reduced, during the driving period, by adding an oversized cutting ring at the forward end of the pipe; but, of course, this increases the volume of soil to be displaced. Tests with vertical piles and undermined caissons indicate that the frictional resistance on such vertical structures is of the order of 600 lb. per sq. ft. of imbedded surface. For horizontally imbedded pipes, a similar value should be expected, except at large depths. The 6-in. lines noted above required a maximum force to overcome a frictional resistance of 405 lb. per sq. ft. of embedded surface, if the displacement factor is disregarded. It would therefore seem sufficient, in planning this type of jacking, to provide equipment and back-stop resistance equal to 600 lb. per sq. ft. of maximum imbedded surface.

Costs. The cost of installing pipes below railroad tracks or developed pavements carrying considerable traffic volume by the jacking method is less than the cost of open cut methods when the backfilling and restoration are included, even if no allowance is made for delay and inconvenience of traffic. Actual costs reported in several of the installations listed in the tables were:

Table 1, Item 16, 60-in. concrete pipe, 96 ft. long in a slag dump fill, $25.55 per lin. ft. total cost (1936).

Table 1, Item 12 and 13, 60-in. concrete pipe, 84 and 88 ft. long, $25.98 per lin. ft. total cost (1935).

Table 2, Item 2, 60-in. Armco pipe, 80 ft. long with 7 ft. cover, showed a labor cost of $5.20 per lin. ft. (1929), and Item 10, 48-in. Armco pipe, 126 ft. long with 34 ft. cover also shows a labor cost of $5.00 per lin. ft. (1929).

Table 1, Item 30, 54-in. concrete pipe, 148 ft. long, showed a labor cost of $4.31 per lin. ft. (1942).

Table 2, Item 5, 36-in. corrugated pipe, 200 ft. long and 66 ft. deep, $24.50 per lin. ft. total cost, half of which was for direct labor in jacking and excavation (1937).

Table 1, Item 32, 54-in. concrete pipe 79 ft. long, in good soil fill, $18.45 per lin. ft. total cost (1934).

CONCLUSION

The procedure of jacking pipe for drainage connections and as sleeves to carry sewer, water, gas or electrical utilities across heavily travelled roadways and through high fills has been developed to a point where costs and construction requirements can be closely approximated. The method should be carefully considered before cutting through an important highway because it will eliminate traffic hazards, delay and accidents so common in the vicinity of detours and construction operations.

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DISCUSSION

HOWARD F. PECKWORTH, *Managing Director, American Concrete Pipe Association*

In practice the concrete pipe manufacturer finds that the use of heavier jacks rather than lighter jacks facilitates the jacking process greatly because of the added speed of operation. In general, in jacking concrete pipes in sizes 36 inches and over, the contractor likes to use two 100 ton hydraulic jacks irrespective of the exact jacking pressures needed.

Of practical importance in maintaining the grade is care in starting the process. This can be facilitated by constructing a timber saddle at the start, long enough to hold two lengths of pipe which are set at grade before the jacking starts. In excavating inside the pipe as the process proceeds it is generally helpful to excavate about an inch outside the outside diameter of the pipe at the top, leaving about an inch of material in the bottom higher than the finished gradient.

The smallest concrete pipe jacking operation on our records is the 18-inch reinforced concrete pipe at West Memphis, Arkansas, and in this case excavation was accomplished by using a 15-inch post hole digger.

The largest concrete pipe jacking operation on our records is the 96-inch Pedestrian Underpass under the R. F. & P. Railroad at the Potomac Yards near Alexandria, Virginia. The top of this concrete pipe is only 2½ feet below the base of rail subjected to extremely heavy traffic.

Since receiving the original

draft of Dr. Feld's paper on October 2nd, I have tried to complete his Table 1 and bring it up to date. This has been most difficult because no attempt has been made by the industry to keep a list of all such installations. The enclosed list to supplement Table 1 is only a small percentage of the total number of concrete pipe jacking installations, a process which is in common occurrence now in every State in the Union.

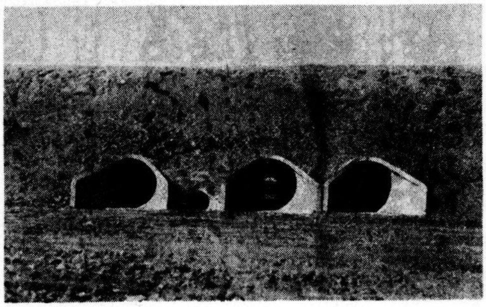
The following enclosures were deemed to be of interest to this paper:

1. Additions to Table 1.
2. Photos showing jacking three lines of 60-inch reinforced concrete pipe at Watertown, South Dakota.
3. Actually jacking pressures as recorded in the article "72-inch Reinforced Concrete Pipe Jacked 144 Feet Through Earth Embankment", by Lionel Pedley from the August 20, 1943 issue of "Southwest Builder and Contractor".

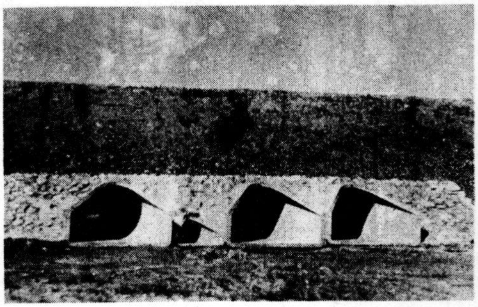
ADDITIONS TO TABLE 1 (FELD)

CONCRETE PIPE INSTALLATIONS BY JACKING METHOD

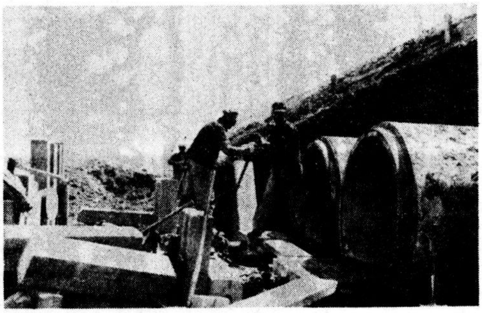
Date	Pipe Sizes in.	Soil Cover ft.	Size of Jacks T.	Lengths Jacks ft.	Cutting Edge	Location
1936	30	3				U.S. 70, Memphis, Tenn.
1936	30			40		Minnesota State Highway
1936	30			40		Minnesota State Highway
1936	30			40		Minnesota State Highway
1937	36			42		Village of Hopkins, Minn.
1937	36			36		Village of Hopkins, Minn.
1937	42			60		City of Minneapolis, Minn.
1937	42			60		City of Minneapolis, Minn.
1941	36			36		Village of Edina, Minn.
1941	72			42		Redwood County, Minn.
1941	42					Northern Pacific RR, Wash.
1943	3-60		2-100	84		Watertown, South Dakota
1945	96	2½		132		R.F.& P. RR, Potomac Yard, Va.
1945	4-48			112		R.F.& P. RR, Alexandria, Va.
1945	72					Renville County, Minnesota
1945	72					Renville County, Minnesota
1946	18		1-50		None	West Memphis, Arkansas Rock Island, Railroad
1946	36		1-50		None	W. Memphis, Ark. Rock Island Railroad
1946	2-36	20			None	U.S. 51, Winona, Mississippi
1946	36			42		City of Rochester, Minn.
1946	36			42		City of Rochester, Minn.
1947	36	15	1-75		None	U.S. 78, Tupelo, Mississippi
1947	36			36		Village of Watson, Minn.
1947	72			42		Freeborn County, Minnesota
1947	42			60		Village of Kasson, Minn.
	33			60		Norfolk, Virginia
	42			60		Franklin, Virginia
	66			60		Franklin, Virginia
	48			120		Newport News, Virginia
	72			40		Portsmouth, Virginia
	72			40		Portland, Maine
	72			80		Paterson, New Jersey
	54			140		Paterson, New Jersey



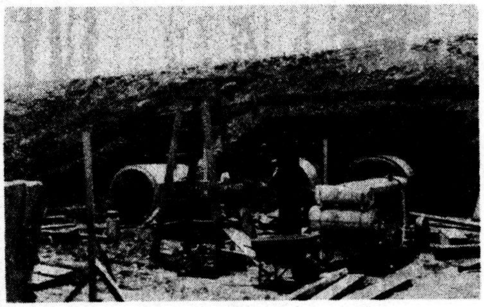
MAY 1943 WATERTOWN SOUTH DAKOTA
Three 60-inch reinforced concrete pipe culverts after jacking was completed (the one 24-inch culvert was placed previously).



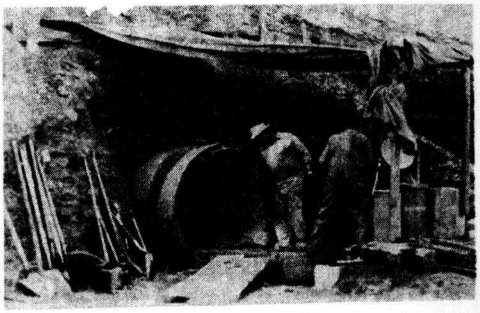
MAY 1943 WATERTOWN SOUTH DAKOTA
Upstream view of the same project.



MAY 1943 WATERTOWN SOUTH DAKOTA
Jacking 60-inch reinforced pipe culverts through railway embankment.



MAY 1943 WATERTOWN SOUTH DAKOTA
Jacking 60-inch reinforced concrete pipe culverts through railway embankment.

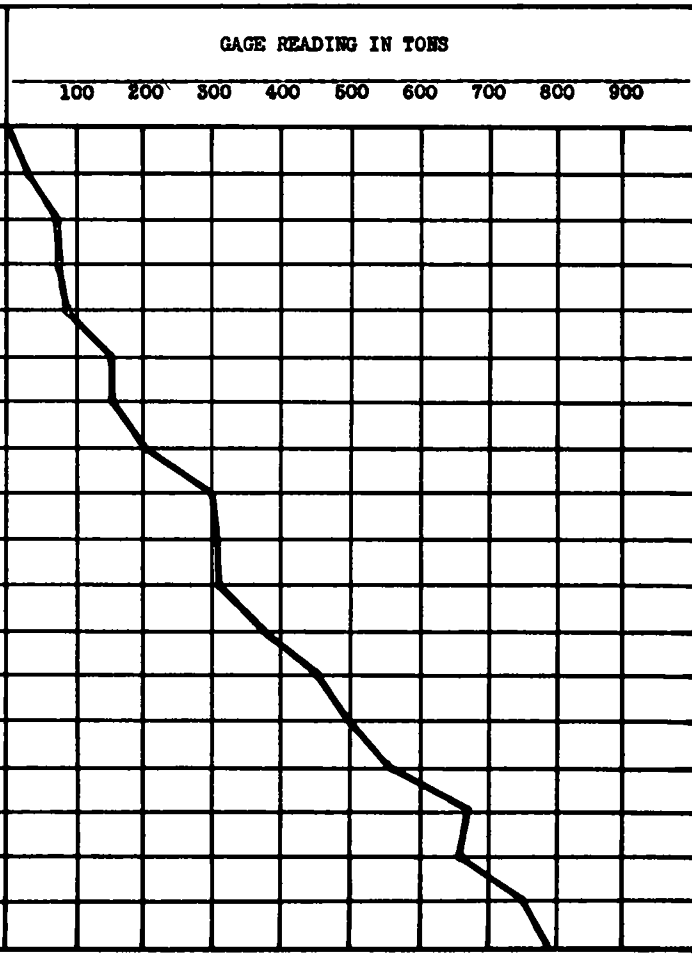


MAY 1943 WATERTOWN SOUTH DAKOTA
Jacking 60-inch reinforced concrete pipe culverts through railway embankment.

72-INCH STORM DRAIN, WILLIAM MEAD HOMES HOUSING PROJECT, LOS ANGELES

Chart showing pressure required to jack eighteen 8-foot sections of 72-inch reinforced concrete pipe 20 feet below four main line tracks of the Santa Fe Railroad. Soil composition: Sand and gravel, moist. Total distance 144 feet. Equipment: Four 200-ton jacks, on manually operated pump. Started August 24, 1942; finished September 30, 1942.

Pipe Sec. No.	Hours @ Sec.	Hours Bet. Sec.
1	32	6
2	23	5
3	18	3
4	17	6
5	23	15
6	31	8
7	52	18
8	22	4
9	28	23
10	24	4
11	28	6
12	45	8
13	42	10
14	51	46
15	47	5
16	67	27
17	60	6
18	83	



GEORGE E. SHAFER, *Chief Engineer Armco Drainage and Metal Products, Inc., Middletown, Ohio*

I would like to take the opportunity to congratulate Dr. Feld on his excellent paper covering the installation of pipes by the jacking method. Not only does this paper represent a lot of Dr. Feld's valuable time but it also brings before an important group like this the fact that it is not always necessary to allow open cutting of important highways for the purpose of installing new culverts, sewer lines, pipe lines, etc. Since safety to the highway traveler is the important issue in highway building and maintenance, one way to help this issue is not to allow the open cut method of installation unless it becomes absolutely necessary.

It is obvious, however, that due to certain soil conditions, diameters, lengths, etc. that the jacking method of pipe installation has limitations. Therefore, because of these limitations I would like to mention another means, or method, of installing pipes or conduits which is used where the jacking method is not practical or feasible. It is the tunnel method of installation to which I refer and this method is becoming more popular everyday with cities and railroads since all work is carried on underground with no interruption to surface traffic.

As mentioned in Dr. Feld's paper, there is a possibility of the pipe "freezing up" due to several contributing factors such as ground conditions, diameter, length, etc. As most of you know, tunnels have been installed under most unfavorable conditions and lengths have not been a limiting factor. Diameters are limited to the ability of supporting materials. However, diameters from 50 in. to 144 in. with which we are most vitally concerned, can be safely supported for unlimited lengths. Supporting material consists normally of tunnel liner plates or timbers.

Tunnel liner plates are made of steel or iron usually varying in thickness from 14 gage up to 3, with the exception of large vehicular tunnels, depending on the loads which they are expected to carry or the expected service life or both. The individual plates are so fabricated that complete erection can be made from the inside. They vary in width from 16 in. to 18 in. and each 16 or 18 in. ring is composed of sufficient plates to complete the shape and dimensions of the desired tunnel cross section. By using various protective devices at the "face" of the tunnel, such as breast boards, shields, poling plates, etc., 18 in. of material is mucked out to allow the erection of a plate. This is continued around each ring until a complete segment of plates is in place. This operation is continued until the desired length of tunnel is completed.

Liner plates are available in various shapes such as circular, elliptical, horseshoe shape, etc., and are used for conduits, drains, underpasses and sewers where it is desirable to install these structures without having to disturb the surface.

To further break down the above uses, the following will give more detail:

1. Underpasses: Underpasses are used by cities, counties, states and industries to allow pedestrians a safe walkway under busy highways, railroads, plants and buildings. They are also used by industry for transportation of products from one building to another where these buildings are separated by highways or railroads.

Armco Drainage & Metal Products, Inc. of Middletown, Ohio, recently published a "Manual of Underpasses and Service Tunnels" which brings out some of the advantages of these structures.

2. Conduits: Where it is not practical to jack conduits for carrying pipelines, sewers or other utility lines, the tunnel method of installation can safely be used for installation of such conduits. In many instances it is possible to put several utility lines in one large tunnel rather than in separate conduits. This could readily result in saving time as well as expense.

3. Drains: a. Under normal conditions a galvanized or galvanized asphalt coated liner plate structure can be tunneled into position thus acting as the finished structure.

b. Under abnormal conditions such as acid soils, a black light gage liner plate can be used to make the necessary hole through the embankment then the desired finished structure can be "threaded" through this hole and the annular space filled with a lean mixture of pressure grout. In this case, the liner plate simply acts as a construction tool.

In numerous instances, due to length or diameter of proposed opening or soil conditions, it has been found more economical to use the tunnel method rather than trying to jack. Consultation with competent men who are familiar with both methods should be helpful when there is any doubt in the minds of the designer or specifying officer.

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