PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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CONTROL OF THE MISSOURI RIVER AT HIGHWAY CROSSINGS IS AN IMPORTANT PROBLEM

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RETARDS IN STREAM CONTROL

Reported by JOHN R. CHAMBERLAIN, Highway Bridge Engineer, United States Bureau of Public Roads [1]

IN CONNECTION with the construction of Federal-aid highway bridges across the Missouri River and other streams the Bureau of Public Roads has had occasion to study the problem of stream control. Where a large and relatively permanent bridge is to be placed across a stream, which if uncontrolled is constantly shifting its channel, the problem is one of major importance. A study of the erosion and silting action of such streams must be made in order that bridge sites requiring the least protection may be selected and that adequate protection may be included as a part of the construction plan. There are cases where the cost of holding a stream to the existing channel under a bridge for a period of years has greatly exceeded the cost of the bridge.

CHARACTERISTICS OF THE MISSOURI RIVER

From Sioux City in northwestern Iowa to the mouth of the river at St. Louis the Missouri has an average fall of 0.86 foot per mile. The valley is flat and lies between bluffs 3 to 10 miles apart, with an average width of about 5 miles.

The land in the valley is mostly of a light soil, easily eroded, and in no place does it lie much above flood stage elevation. Bed rock is mostly from 50 to 100 feet below low water and there is little variation in the character of soil to this depth.

The stream carries great quantities of silt in suspension. In August 1923, when unfiltered water from the river was pumped into the Omaha distributing system it was said to contain as high as 43 per cent of silt. This was, of course, considerably above the average.

The stages of the stream are usually referred to as standard high and standard low elevations. These stages are the average high and low for the period under observation prior to 1888. The difference between standard high and standard low at Sioux City is 10.42 feet; at Kansas City, 14.52 feet; and at St. Charles, 16.14 feet. The flood discharge at Sioux City at standard high stage is about 200,000 cubic feet per second and at St. Charles, 300,000 cubic feet per second. The flood of 1892 discharged 650,000 cubic feet per second and that of 1903, 750,000 cubic feet per second at St. Charles.

The width of the stream between banks or standard high water contour varies from about 1,000 feet to more than a mile in places. Its depth at low water is insufficient to float a boat of 4 feet draught though as great a depth as 65 feet has been observed at flood stage in certain places.

The stream, in general, meanders back and forth in the valley from bluff to bluff and, by reason of the rather extreme fall and the instability of the soil it effects, when not controlled, rather pronounced changes in location by continuous erosion of its banks. Maps of this stream made to-day superimposed on maps made prior to 1890 show in some places, particularly in the Dakotas, such erratic changes that no relation can be seen between the location of channel now and then. Figure 1 shows a section of the river below Omaha, Nebr., as it existed prior to 1890 and in 1898. Parts of the river as it is to-day are also shown. It is not unusual for erosion to change the lines of the stream as much as a half mile in a single year.

PROCESSES OF EROSION

Changes in channel occur by overflowing through a swale and thus developing a secondary into a principal channel, and by lateral erosion. The former is the most spectacular but the latter is the cause of the greatest concern. Sometimes at flood stage the overflow will cut across a large horseshoe bend and will erode a channel of river proportions in a very short time and by such action shorten its length many miles. Such a major shortening of the stream gives rise to far-reaching effects. By increasing the slope it produces higher velocity in the stream both up and down for many miles. There also follows a change in the oscillation of the current between banks. The result is excessive lateral erosion tending to cut more and deeper bends. Many such major cut-offs have taken place in

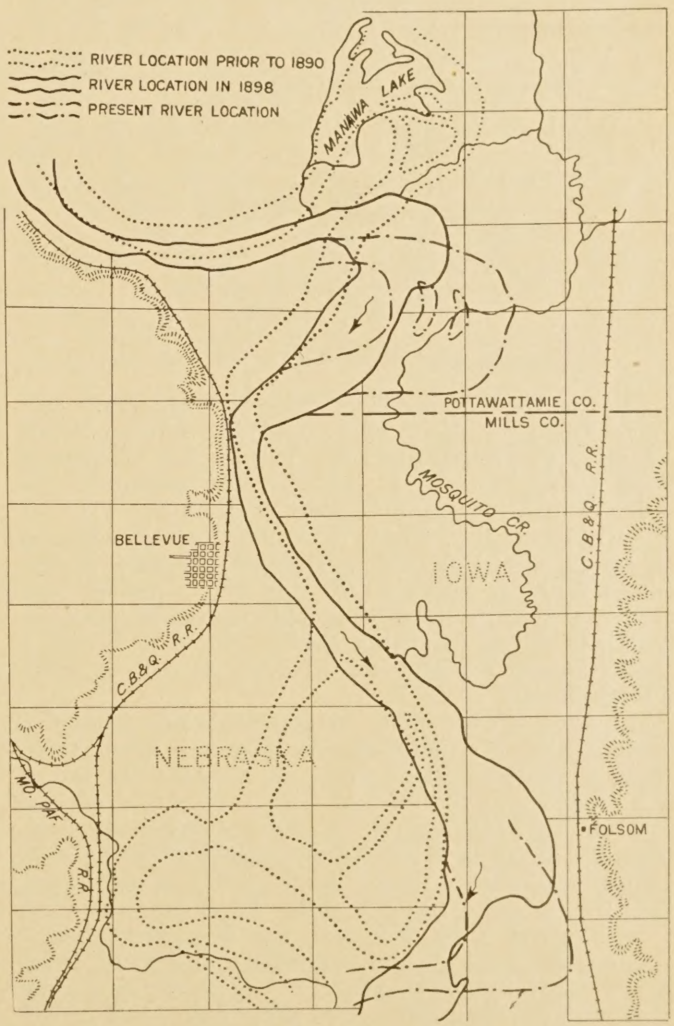

FIG. 1.—Section of the Missouri River below Omaha, Nebr., showing pronounced changes in channel from about 1890 to the present time

[1] This report was prepared by Mr. Chamberlain a few months prior to his untimely death on Dec. 15, 1925.

recent years and the stream has corrected itself by lateral erosion so that its total length between distant points has remained practically unchanged.

In general the river is a series of bends first to the right and then to the left. In these bends the water is deep along the concave bank and unless the concave bank coincides with the rock bluff at the edge of the valley or is protected by artificial means the shore line yields to erosion.

Even in places where the general direction of the river is straight for a considerable distance, lateral erosion may develop if conditions have been such as to form a bar in midstream, a condition which will normally occur where the river is wide or just below a reach where velocity is great. The bar in the path of the current has the effect of crowding the stream to the sides where

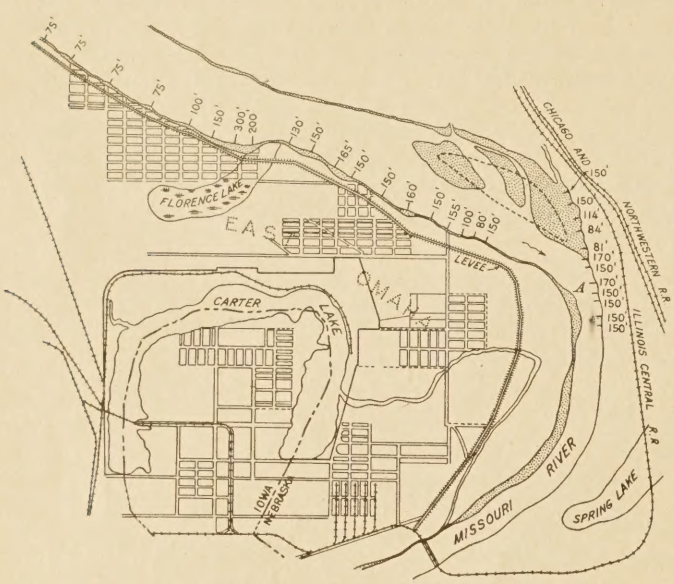

Fig. 2.—Map showing location of retards at East Omaha, Nebr.

it attacks the banks. This was the condition just above Glasgow, Mo., when extreme erosion made expensive protection necessary to save the Chicago & Alton railroad in 1924. It was also probably the principal cause of rapid cutting of the north bank at Waverly in 1923. Instead of the bar itself yielding to the attack of the current, presumably because it presents a taper or wedge to the current, it deflects the stream without itself suffering dislodgment of material. Examples of this condition may be noted in Figure 2 where protection work is shown along the convex shore at East Omaha.

At points where bank erosion is severe, excessive depths are usually found and the erosion is most active during a receding stage, probably because the soil is then soaked and the ground water presents a reverse head tending to dislodge the particles of soil in the bank. As great a depth as 65 feet has been observed along such eroding banks at a point about 100 feet off shore. With this in mind the difficulties of placing any kind of construction to stop erosion can be appreciated.

As a bend develops and embodies a change of direction or central angle of near 180° there comes a time when the fall round the convex shore line is sufficiently greater than around the outside or concave shore line to cause the major part of the flow to follow the shorter passage and thus lessen the current near the concave bank. This is referred to as chord action, as the current flows in the direction of a chord and, continuing in a straight line across its former channel, impinges directly against the outside bank. When this occurs it presents a difficult problem, in bank protection. Such a condition exists near East Omaha the current impinging against the bank at the point marked *A* in Figure 2.

METHODS EMPLOYED IN STREAM CONTROL

There are two general methods used in the control of streams. The first is to change the flow by directing the current away from the eroding bank in a desired direction. The other is to accept the current as found and make the bank safe against erosion. The two methods are sometimes combined by retarding the current along the eroding bank and at the same time partially deflecting it away. As an incidental result a deposit of silt is formed on the downstream side of the obstruction used for retarding and this in turn results in building out the bank to some new shore line.

Where the river lies close to a rock bluff or edge of valley or where it can be made to do so the scheme of so deflecting the current that it will maintain such a position is often a desirable undertaking and if the alignment of the rock bluff is straight or slightly concave, it becomes a relatively easy task to train the stream by deflecting dikes. This practice is particularly recommended for bridge crossings of streams that must be kept open to navigation.

Where the stream forms a bend in mid valley and the bend is relatively smooth and on a flat curve that gives promise of future stability if maintained the second method is ordinarily best adapted.

Retarding and partially deflecting the current appears to be the method most commonly used by land owners presumably because it is the only scheme that lends itself to minor operations. If permeable dikes are introduced at intervals along the shore line, the water is retarded and if the velocity is slower after having passed the obstruction, it follows that the excess must be deflected out around the end and this deflection of current tends to change the direction of the stream at that point. Thus if a prism of water approaches the obstacle with a velocity of 4 feet per second and that portion which passes is reduced to 2 feet per second one half of the flow will of necessity be deflected. It also follows that if the space behind the obstruction is eventually filled with silt, the only remaining function of the obstruction is to deflect.

Making the bank proof against erosion has the advantage over the deflection of current in that it precludes the possibility of injuring other property owners along the stream.

If work is installed on a piecemeal basis or in isolated projects instead of over long reaches, its success is threatened by changes that may occur up stream no matter what methods are used. If the work has been placed to meet conditions as they exist, such as the protection of a bend, and if the point of attack of the stream changes so that it will be further up stream, as might easily happen, by say the formation of a hook in the opposite shore line in a bend above, then, independent of the type of construction, the work installed must fail. Or should conditions develop so that the current makes a chord across the bend an especially difficult condition appears which the work may not be prepared to withstand.

These features make it necessary to revise our ordinary concept of permanence of construction and emphasize the fact that work of this class should be thought of as a continuous operation and not first construction and then an annual percentage for maintenance.

DEVELOPMENT OF METHODS OF RIVER CONTROL

The major portion of river control work on the Missouri River has been installed by the War Department in its effort to make and maintain a navigable channel. Detailed description of such work is to be found in the files and reports of the Missouri River Commission and the War Department.[2]

Retarding the current with trees or saplings, one end of each anchored to the bottom with stones and the other end kept afloat by a buoy, was about the first plan tried. The idea was borrowed from India where it was employed in rivers not subject to ice. It produced the expected result but lacked durability, principally on account of damage by ice. This method was introduced on the Missouri River 45 years ago and various modifications were tried, such as wire instead of brush and tripod instead of stone anchors. The evolution of this method resulted in a type of permeable pile dike still in use and consisting of two or three rows of piling framed together for lateral bracing and supporting a curtain of vertical poles spaced closely to more effectually retard the current. Around the piling is spread a woven brush mattress anchored down with rock. This mattress is 75 feet wide and extends downstream from the dike a distance of 50 feet.

The dike is placed so as to make a slight angle with the normal to stream flow. It is built from the bank out to the desired new bank. An objection to this form of dike is its tendency to catch drift, which, if accumulations are great enough, may lead to its destruction. To serve the purpose of creating a new

FIG. 3.—Middle dike of a series of three pile dikes protecting bridge at Leavenworth Kans. Note accumulation of drift

bank it must be built above low water and hence the piling is subject to decay. The cost of this type of dike is in round figures about $25 per foot. While it has been used on concave banks it has not proved sufficiently substantial to justify its general use in such places, particularly where conditions are severe. Figure 3 illustrates a pile dike with drift accumulation at Leavenworth, Kans., and Figure 4 illustrates another installation with a brush mattress.

[2] A detailed description may be found in the transactions of the American Society of Civil Engineers, Vol. LIV, written by Mr. S. W. Fox, for many years principal assistant to General Suter of the Missouri River Commission.

This type of protective work was successfully employed prior to 1923 on the convex bank opposite Waverly, Mo., for the purpose of forcing the river over against the south bluff and holding it there. At that time a relatively minor influence started the current toward the dike-protected shore. The dikes proved entirely inadequate to stop the very severe erosion and one dike was practically destroyed. Work of restoring the dikes to perform their original function is now in progress.

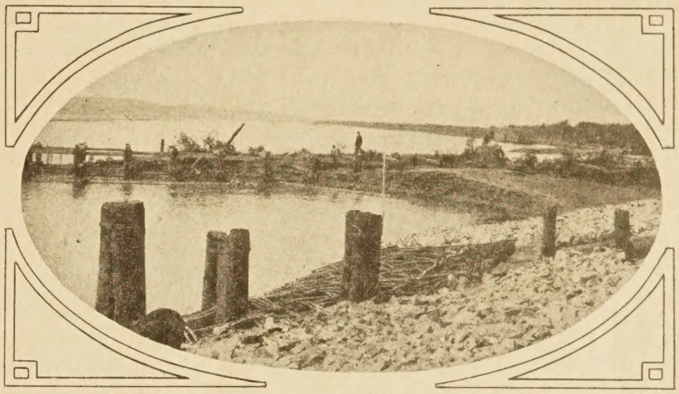

FIG. 4.—Mattress protection for new pile dike. Old dike in center of picture being replaced

BANK HEADS, LONGITUDINAL DIKES, AND ABATIS

During the early nineties experiments were made with bank heads, longitudinal dikes and a type known as the abatis. The bank heads were formed by paving the banks at intervals with mattresses of brush and stone extending well down to the possible depth of scour. They were built so as to form segments of circles in plan view. The thought was that they would so deflect the current that, if placed at frequent intervals, no erosion would occur between them. The radius was about 316 feet. This particular length was arrived at from experiments and calculations having in mind the effect of eddy currents on the downstream side. Several of these structures were built for protection of concave bends. It was found, however, that they did not prevent large bays or bights forming between them. They soon gave evidence of failure and the last was destroyed in the flood of 1903.

A case that is not unusual is that in which it is desired to correct a bight or pocket in a concave bank. Such pockets develop rapidly into large and deep bends which are more difficult to maintain and which also effect a change in the regimen of the stream further down. Longitudinal dikes have been employed to meet this condition by building the dike along the desired shore. The first longitudinal dike on the Missouri was built in 1896 just above Omaha across a bight or bay and along the line of the desired shore. It was 2,600 feet in length tied into the bank at each end, and had stem dikes placed at right angles connecting the main dike with the shore at intervals. The main dike consisted of three rows of piles with a mattress 125 feet wide, 100 feet of which lay out in front of the dike on the river bed. A second mattress was attached to the upper 750 feet of the dike supported upon a wale of the dike some 3 feet above low water. This mattress sloped from the dike outward so that when the space behind the dike had silted in the supported mattress would lie upon the newly made bank. This was said to be successful and later four more were constructed; one at Nebraska City, one at St. Joseph, one at Glasgow and one at St. Charles. The later form consisted of five rows of piling with tops 3 feet above high water. The supported mattress only was used and it sloped from the center row of piling outward and also toward the shore in long flat slopes. These installations were successful but further use of the type was not made on account of the cost which was $35 per foot at a time when common labor cost $1.10 a day.

The abatis, a type of construction named after the military device of similar shape was looked upon as a cheap form of construction to be used in closing chutes and advancing shore lines where full force of the river would not impinge against it. It consisted of triangular-shaped frames supporting longitudinal timbers which, in turn, supported poles or plank.

Bank revetment work was first used on the Missouri River in 1880. Figure 5 shows the type of wire and brush mattress finally arrived at after many years of experience. It is regarded by some as the most permanent and reliable scheme of river control. Unlike the dikes it is not subject to decay since all wood is below water; it offers no interference to drift and is not affected by ice. It accepts the river where

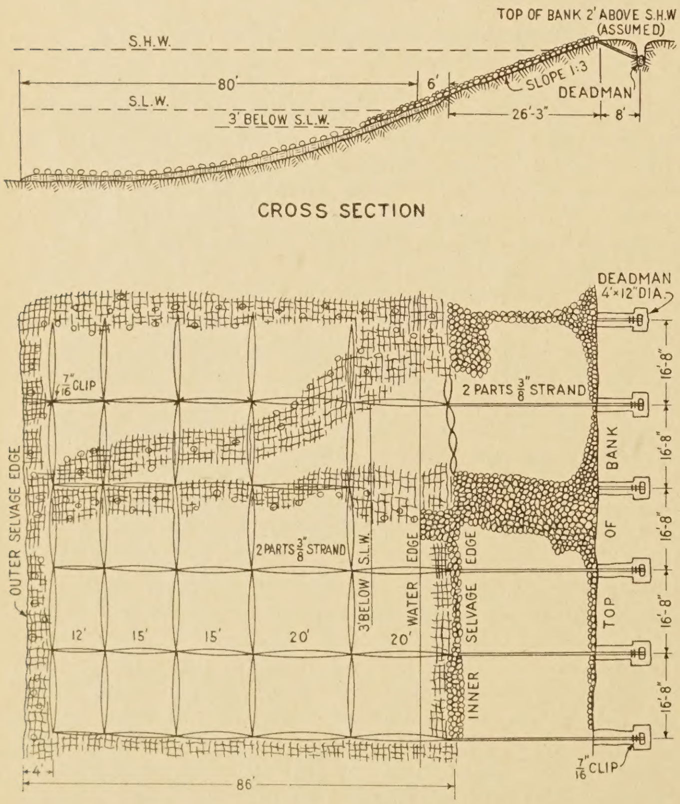

FIG. 5.—Standard form of bank revetment used on the Missouri River

found, hence does not of itself produce effects either favorable or unfavorable at other points. While it is ordinarily thought of as permanent its length of life is considered good if it holds for a period of twenty or twenty-five years. The revetted bank above Glasgow has failed in stretches more than a half mile long under attacks which were apparently not very severe. A revetment at Council Bluffs at what is known as the Narrows has failed but under most extreme punishment.

Various methods of constructing and using mats have been suggested. Perhaps the most noteworthy is credited to the Wabash Railroad. This type is known as the Cunningham mat and consists of willow poles or brush laid in two directions forming a mattress about 1 foot thick which is held in position by wire fencing laid beneath and on top of the brush. The lower fence wire is tied to the upper strands at intervals of

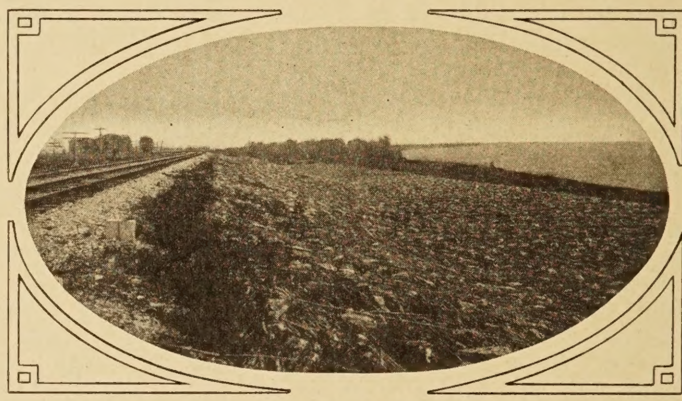

FIG. 6.—Cunningham mattress 100 feet wide protecting the tracks of the Wabash Railroad near Missouri City, Mo.

about 3 feet in each direction and enough rock fragments are laid beneath the upper layer of fencing to cause the mattress to sink. This mattress has been installed by the Wabash Railroad near Missouri City, Mo., and is said to have been successful. As the work appears to-day, much of the mattress is submerged but a larger area is spread out along the bank as safeguard against further encroachment of the stream. Figure 6 shows this construction.

An objection to this type of construction is that the galvanized wire fencing has a limited durability, and if laid upon the bank as practiced by the Wabash Railroad, and the river does not undermine it as intended for a period of one or more years the willow brush becomes too brittle to be of much service when once submerged unless by good fortune it drops quite gently without tendency to break up. It is said to cost at the present time, including a patent royalty, approximately $20 per running foot, 100 feet wide.

The Missouri Pacific Railroad makes frequent use of a type of mattress which consists of a single layer of wire on which willow brush is laid and, over the brush and at right angles to it, heavy poles which are wired down to the lower layer of fencing. Upon this mattress, directly above the poles, is placed the rock ballast required for sinking. The cost of this type of construction is unknown but is probably not greatly different from that of the Cunningham mattress.

RETARDS ANCHORED TO SHORE

Tree retards have been used for many years by landowners in their efforts to stop the caving of banks. At first the trees both large and small, were simply thrown into the stream and anchored to dead men on the bank. Such an arrangement does give some protection but with water more than 20 feet deep it is not conceivable that the effect of floating trees so anchored, could be great. Another form of retard that has been used consists of fastening three poles approximately

16 feet long at the center so that the assembled poles form a unit as illustrated in Figure 7. Wire is then strung around the unit connecting the ends of the poles so as to form a wire entanglement. Several of these units are joined together by a cable, one end of which is anchored to a dead man on the bank, and the whole series is then rolled into the stream. These series are spaced at desired intervals, usually very close together so that the mass in the stream will form a practically continuous entanglement. This type of retard was first tried on the Kansas (Kaw) River where it is reported to be quite successful. In that stream, where installations have been seen, the entanglements partially buried themselves in the streambed, and the protruding tops caught and accumulated floating drift. Their shape and anchorage made it possible for them to hold fast and thus effect a rather substantial mass. In the Missouri River the greater depth of water is against their catching drift and unless they do in fact silt in their presence apparently becomes a cause for more active erosion due to the eddy action around individual timbers where they are in contact with the bottom of the bank.

In the Niobrara River in Holt County, Nebr., which has a wide, shallow, swift current, they proved a failure due to this cause. The poles in that installation were structural steel angles and they sank almost straight down so that their effect could not be observed within a few months.

RETARDS ANCHORED TO STREAM BED

Trees make particularly satisfactory retards if substantially anchored to the bottom because the current then forces the trees to the bottom. A concrete pile driven a few rods upstream makes an effective anchorage, the stability of which, however, is dependent upon the possibility of placing the anchorage deep enough below the stream bed to insure against displacement by the pull on the cable. While this

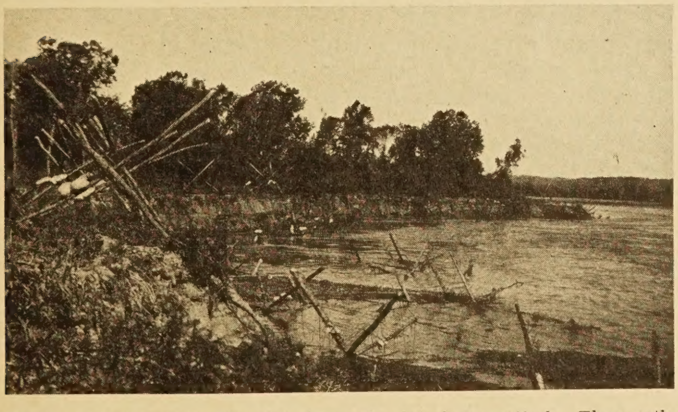

FIG. 7.—Jack-stone jetties placed in stream at point of severe attack. Those on the bank indicate the method of construction

feature is not ordinarily a factor it can be easily understood that with rock-bottom less than 70 feet deep and a depth of stream of 50 feet or more the problem is of importance. If the structure stands the first flood attack, the probability of its loss is lessened since such silting as generally occurs is an added safeguard against future dislodgment.

A factor to be guarded against is the formation of eddies on the downstream side which is likely to occur if the current is especially strong or where it does not nicely parallel the shore line. Another feature which effects the permanence of this type is the breaking of cables due to the whipping of the trees. Places have been noted where the trees were still in motion two years or more after installation. It is of course a severe test of a cable to withstand the bending and twisting that goes on while the trees are continually changing their position.

In the construction of these retards the trees are piled in cordwood fashion on the bottom of the stream

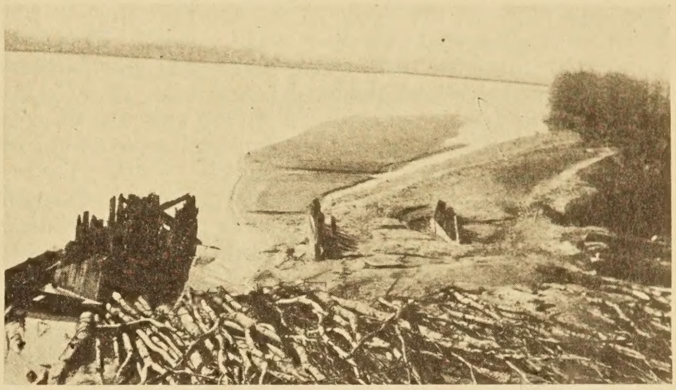

FIG. 8.—Looking downstream at tree retards from Illinois Central bridge at East Omaha. Butts of trees reversed on account of eddy action

until they show above the water surface at ordinary stages. They are laid with butts upstream and each tree is fastened by cable to the anchor cable. The concrete pile anchors are placed approximately 35 feet apart and about 100 feet upstream from the ends of the trees. The trees near shore are piled well above the high-water stage and are anchored by cables to dead men back from the bank line. Units or piles of trees are spaced along the bank at varying distance depending mostly upon the length of the unit and upon whether the current is parallel to the shore line or approaches at an angle. The usual spacing may be taken as 500 to 1,000 feet for 150-foot units.

The piling used for anchorage is about 15 feet in length and 16 inches square and is sunk under its own weight by the jetting process. The jet pipe is in the center of the pile with a device for disconnecting the hose supplying the water after the pile is in place. The trees are put in place either from a boat or from land. If a boat is used, it is anchored so that when the trees are skidded off they will take the desired position in the retard. This is done by anchoring a pulley block on shore and the pull required to skid the trees is applied from a capstan on the boat.

When trees are placed from land a temporary pile is first driven in the stream at the proposed end of the retard. To this pile is attached a pulley block and a hoisting engine placed well back on the bank furnishes the power for pulling the trees in groups of a dozen or more outward to their final position. Figure 8 shows a tree retard of this character near Omaha.

RIVER PROTECTION AT EAST OMAHA

Figure 2 shows a map of the river just above the city of Omaha. At this point the river crosses the valley from the west bluff to the east and then returns again to the west bluff and the land inclosed by this bend is called East Omaha. The area is mostly farm land but numerous houses and cottages are built in the vicinity of Carter Lake. A levee has been constructed to protect this area from overflow and in 1922 the eroding bank threatened to intercept this levee. To prevent this a series of 19 retards, covering a distance of 3½ miles of shore line, was installed. The cost per foot of retard was about $50 which amounted to $8.20 per foot of bank protected.

The general direction of the stream is straight but there exists one quite pronounced hook in the shore line and for some distance below this hook the bank is, in fact, convex. While the map does not so show, the opposite side of the river is so shallow that a bar appears during ordinary low water stage. This bar appears to force the deep water along this shore and so maintain it even where the bank is convex. Conditions at this point are not considered severe and the retards were successful in stopping erosion and have silted in irregularities as illustrated in Figure 9.

In Figure 2 it will be noted that the Illinois Central Railroad lies quite close to the river near the existing bank which was of sand. The bank at this point was first revetted but the revetment failed and the construction of tree retards was started in 1922. Construction was begun at the upstream end of the series shown at this point. After the construction of the first retard and while the others were under construction the sand bar shown on the concave bank formed and chord action of the current, though not a pronounced case, started. The current has been thrown away from the bank where it was nearest to the railroad and now impinges with considerable force against the third, fourth, and fifth retards from the lower end. These structures have held the bank but strong eddy action takes place in the bights between them. Additional trees have been thrown in these bights and anchored to dead men on shore. The trees have been whipped about violently by the whirlpools but at the time of inspection appeared to be holding on very well.

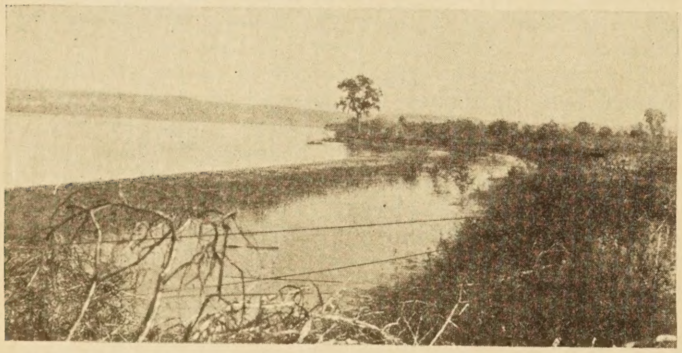

FIG. 9.—Silt deposit resulting from retard construction at East Omaha. Picture taken from tree retard a portion of which shows at lower edge of photograph

The map shows two bar outlines, the position in April, 1923, being shown by a full line and the position in October of the same year by a dotted line. At the latter date a considerable volume of water was passing close to the shore where the bar had been a few months before. In 1924 this channel again practically closed up. The stream appears to be in a state of delicate balance at the bend, so that at times the current takes the outside of the bend and at other times the short course, with resulting chord action.

At Gibson, Nebr., the Chicago, Burlington & Quincy Railroad constructed retards to protect its yards. These retards have become covered with rubbish dumped into the river and have caused the formation of a bar directly downstream. With the retard covered it seems that the current would cut away the bar. Instead of doing this, however, the bar deflects the current to the opposite bank below which is being eroded. The river at this point is quite straight and the deflection of the current by the bar is an excellent example of wedge action.

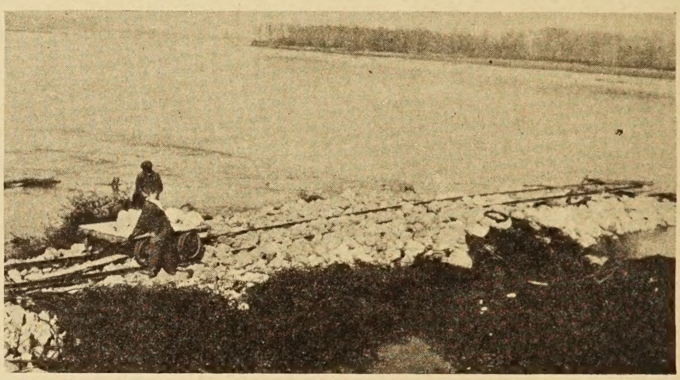

FIG. 10.—Stone and brush dike under construction showing silting that has resulted

GENERAL OBSERVATIONS ON RIVER CONTROL

An ideal plan for maintaining a river such as the Missouri in a permanently fixed channel would require holding it to a nearly uniform width at all points, shaping all bends with a maximum radius and developing as much total length as possible. This would require the protection of the outside of bends when not in contact with bluffs and the installation of structures on the inside of bends to prevent chord action. The advantage of controlling long stretches of the river as a single project is illustrated by the work of the Missouri River Commission on 45 miles of channel between Jefferson City and the mouth of the Gasconade from 1892 to 1902. This work cost about $2,500,000 or $55,000 per mile and is said to have required little maintenance since. The total amount is only about twice as great as the Burlington Railroad is said to have spent on a single bend near Folsom, Iowa.

Much of the work of river control has been done by landowners, towns, or railroads in an effort to protect their property. Generally this work has been limited to the immediate vicinity of the danger and often has been delayed until the danger has become acute. This has led to increased costs and sometimes to complete or partial failure.

For the highway-bridge engineer planning the location and protection of bridges over such streams as the Missouri River there is available the results of many years of experience in such work and he must avail himself of it if he is to construct a permanent structure.

A STUDY OF UNUSUAL EARTH ROAD CONDITIONS IN NORTHEASTERN IOWA[1]

By QUINCY C. AYRES, Associate Professor, Iowa State College

SOFT spots and mudholes which developed after grading in a number of earth roads in northeastern Iowa manifested such unusual characteristics both as to their behavior and their location that the writer was engaged by the State highway commission to conduct a field investigation of the causes of their occurrence.

In nearly all cases the soft places did not exist before the grading of the roads. Yet after grading they began to develop at the crest or along the slopes of nearly every cut despite the fact that ample precautions had been taken to insure adequate drainage in the customary manner.

Unlike the wet spots caused by side-hill seepage these failures are seasonal in character, breaking out only in the spring when frost begins to leave the ground and continuing to cause trouble until some time after the frost has disappeared. During this period they become saturated with confined water and are no more capable of supporting a load than deep beds of soft putty which they closely resemble. The occurrences are interspersed at frequent intervals between stretches of excellent roadway which bring them out in sharp contrast and render them all the more exasperating to the traveler. Certain other peculiarties have been observed, such as a tendency of the minor failures to shift position slightly from year to year, and the strange effect of rainfall which seems to improve their condition. The most plausible explanation of the latter phenomenon is to be sought in the temperature of spring rains, which are usually warm enough to melt the frost and open percolation channels.

Natural relief can not be expected until the surface evaporation becomes sufficiently rapid to dry out the top 4 or 5 inches of the road, forming a hard, tough crust. This crust, when once formed is generally thick enough to distribute loads and bridge over the soft material beneath until the following spring. At no time, however, is the crust capable of supporting heavy loads without a perceptible sag similar to that of a steel rail under the wheels of a locomotive.

Clearly these soft spots constitute a problem of a special nature which requires particular treatment for its solution. The objects of the investigation were therefore (1) to discover the cause of the trouble, (2) to determine the most feasible remedy for existing failures, and (3) to find the best way of handling future improvements so as to prevent their recurrence.

So far the occurrences have been observed only in nine counties in northeastern Iowa in the vicinity of the Mississippi River. Clayton County seems to be most seriously affected, but a number of failures have been noted in Allamakee and Dubuque Counties, and to a lesser extent the counties of Winneshiek, Fayette, Jackson, Jones, Delaware, and Clinton have also been troubled.

In Clayton County alone, the total length of the failures, in improved primary roads only, is more than 8,000 feet, and if this figure be increased in the proportion that the present improved mileage bears to the total mileage of primary and county roads in the county, it appears as probable that something like 32,000 feet of roadbed will eventually require treatment.

CONCLUSIONS AS TO THE CAUSE, PREVENTION, AND REMEDY OF THE FAILURES

As a result of the writer's investigation, in the course of which 444 test borings from 4 to 21 feet deep were made in roads in Clayton County, the following conclusions have been reached with respect to the cause, prevention, and remedy of the failures.

The soft spots have been found to occur almost exclusively at the crest or on the side slopes of cuts made in grading the roads, and the investigation indicates that they result from the exposure by the grading operations of unweathered, loessal clay which, in the unimproved road, was overlain by stable, weathered material. The water contributing to the condition is, in the main, of purely local origin. Underground sources of supply, such as springs or seepage veins, have, as a rule, been definitely eliminated.

For the curing of the existing failures the most practical remedy, in the opinion of the writer, is to remove the unstable material to a depth of at least two feet and replace it with well weathered topsoil or "black dirt," thus providing a stable crust to bridge over the unstable material during the critical period, and duplicating the condition known to exist at places where no failure has occurred. Several other remedies have been proposed, among them the covering of the affected areas with rock or sand, the addition of lime to the soil, the use of tile drains, the planting of trees which it is hoped will remove the moisture from the soil, the burning of the unstable material, the paving of soft places, and others of a less practical character. For reasons which will later be presented the writer believes the formation of a crust of weathered earth to be the most practical procedure; but it would perhaps be wise to test a number of the proposals which seem to be feasible with a view of adopting the one which proves to be the most effective and economical.

For roads to be graded in the future two methods are proposed in order to prevent the creation of the faulty condition: Either, (1) adjust the grade line to avoid cutting the hills by making heavy fills with borrowed earth; or (2) balance the cuts and fills in the usual way by making allowance for the extra depth of excavation in cuts necessary to provide for replacement of unstable material with weathered soil.

EXTENT AND PROCEDURE OF THE INVESTIGATION

The area covered and the roads studied in the writer's investigation are shown in the map, Figure 1. The evidence necessary for the determination of the cause and character of the failures was obtained by boring test holes to reveal the nature and thickness of the various strata.

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[1] This article is substantially identical with a paper presented by the author at the annual meeting of the Iowa Engineering Society at Mason City, Iowa, January 27, 1926. It is based on an investigation conducted by the writer between July and September, 1925, and upon preliminary examinations by Mark Morris and M. L. Hutton, of the Iowa Highway Commission.

The first test hole was dug at station 875+00 of primary road Nos. 10–13, about 1.5 miles southwest of McGregor (fig. 1) from which a continuous line of borings was extended to a point near Elkader at station 165+00. Two hundred and sixty borings were made in this section of graded highway, each of which penetrated the loessal deposit and extended into the till or residual soils beneath. For the most part, the holes were dug on the left shoulder of the road about 8 feet from the center line and the usual depth was from 10 to 15 feet, extreme variations being from 4 to 21

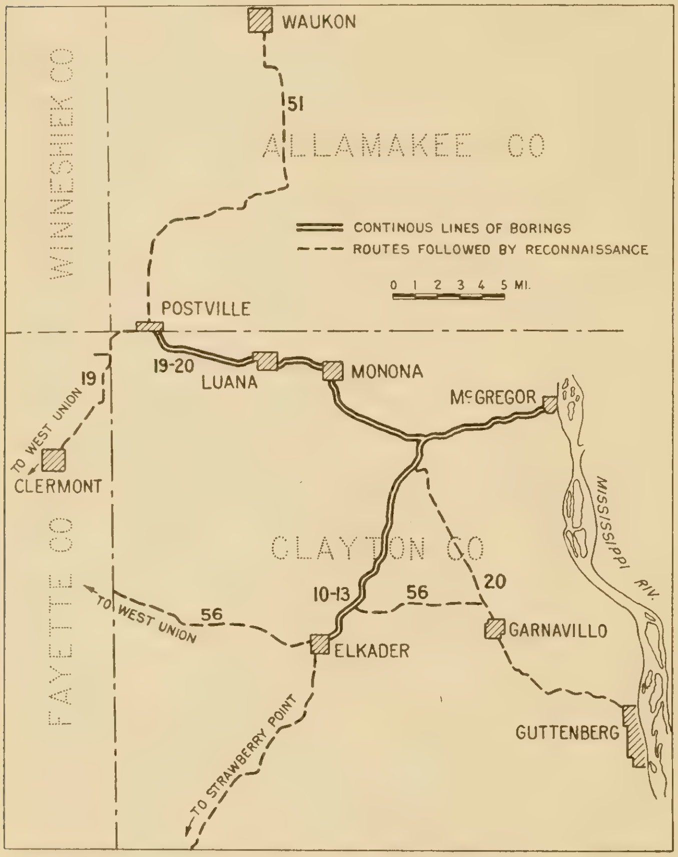

Fig. 1.—Map indicating continuous lines of borings and reconnaissance routes

feet. The diameter of all the holes was 6 inches and the spacing in all cuts was from 50 to 100 feet or closer. A number of holes were also dug in the stable portions of the road, and a few were located in adjoining fields. At the time of making each boring depth measurements were taken and every change in color, texture or consistency of the soil was recorded.

The station number of each hole was accurately determined from the permanent reference hubs and a continuous line of levels was run to establish the elevation of the road surface at the holes. The elevation of the top of both banks opposite each hole was also obtained so that original surface elevations could be computed.

In this way every cut in the McGregor-Elkader road was thoroughly investigated and an attempt was made to account for those that had not caused trouble as well as for those that had. The failures of 1925 which, on this section of highway, aggregate 3,933 feet in length, were identified by laths driven in each bank opposite their extremities, and wherever such a stake was found its station number was recorded and used to locate the borings. A few unstaked failures were located from the records of County Engineer Hahn.

The next stage of the investigation consisted in boring all the proposed cuts in primary road Nos. 19–20 from Postville to Monona, at present unimproved, though the plans for such improvement are complete. One hundred and forty six holes were sunk in the manner already described except that in this road many of the borings were made along the center line in relocations. When the notes of these borings have been plotted, it should be possible to predict with a fair degree of accuracy the location of the failures that may develop after construction.

For the purpose of securing corroborative evidence, 41 holes were bored, covering the eight failures between the intersection of primary roads 10–13 and 19–20 and Monona. These failures, which have an aggregate length of 1,939 feet, were obtained from County Engineer Hahn.

In order to make certain that the soil conditions revealed by the continuous lines of boring were truly typical, the writer conducted brief reconnaissance examinations of the following roads:

Primary No. 51—Postville to Waukon.
Primary No. 19—Postville to West Union.
Primary No. 56—West Union to Elkader.
Primary Nos. 10–13—Elkader to Strawberry Point.
Primary No. 56—Elkader to Garnavillo.
Primary No. 20—Intersection of primary Nos. 10–13 to Guttenberg.

All cuts on these roads, including a number of treated and untreated failures, were closely inspected and sufficient borings were made to identify the various strata as belonging to the classification previously established from the continuous lines of borings, as shown in Figure 1. Nothing was found to controvert any of the conclusions previously indicated.

CAUSE OF FAILURE

Early in the progress of the investigation it became evident that the difference in elevation between the road grade and the original ground surface was the key to the situation and not the depth of cut below the old road, unless the two happened to coincide. Proceeding on this premise, the cause is to be sought in the geological and soil characteristics of the area affected.

Without going into an extended discussion of geologic history, this region may be said to lie in a position untouched by the last glacial invasion. Consequently it is blanketed by a layer of fine-grained, loessal clay (presumably of wind-blown origin) which covers the tough sandy clays laid down in a previous glacial epoch or residual soils composed of weathered rocks. (See fig. 2.) These underlying clays though stiff and gummy are mixed with sand and angular rock fragments which render them permeable and firm. In color they range from a deep, ox-blood red to pale tan and wherever exposed, provide an excellent roadbed.

The overlying blanket of loessal clay on the other hand, never contains any sand or rock fragments and, in general, presents opposite characteristics. Its usual thickness is from 9 to 15 feet and, though somewhat thicker on hills, as a rule it conforms closely to the residual subsurface. In its undisturbed state, three degrees of weathering are clearly marked, as illustrated in Figure 2.

First stage of weathering.—Starting at the bottom (on the residual subsurface) the first stage is noted as a pure, bright, gray clay interspersed but not mixed with thin laminated streaks of dark brown and yellow oxides and carbonates of iron (limonite striations). This clay, though fine grained, is rather stiff and requires considerable pressure to work in the hands. Its consistency in place is generally that of stiff putty, but, when dry it is powdery and fluffy, like flour. In some instances, where it overlies colored sandstone, a thin layer at the bottom of the stratum is found to be discolored to chocolate-brown, mouse-gray or ink-blue.

Second stage of weathering.—In the second stage, immediately above the first, a dull, drab gray occurs mixed with nodules of brown and yellow limonite, which has been partially oxidized. The mixing process is not so pronounced near the bottom of the layer but it gradually increases toward the top, until, in the uppermost portion, the mixture is so well mottled that its component parts are difficult to detect. In its natural condition, the clay in this stage is generally quite wet, ranging in consistency from very soft to soft putty, and occasionally stiff putty. When worked in the hands it is very soft, yielding, sticky and plastic.

Third stage of weathering.—The third and last stage represents a stratum of the same mixture in an advanced state of oxidation lying near the ground surface. It is no longer possible to discern particles of limonite nor is any of the gray clay visible. The product of complete weathering is a fine-grained clay of even texture and uniform color, ranging from a light buff-brown at the bottom of the stratum to a darker brown cast near the top. In fields, the top 18 inches or more is permeated with humus and decayed vegetation, which is responsible for the dark color and its familiar name "black dirt."

PIERCING OF WEATHER LINE CAUSES SOFT SPOTS

Weather line of the third stage.—Before considering consistency, it is well to divide this upper stratum into two parts: (1) Stable; and (2) unstable. One can never be certain just exactly where this line should be drawn, even while making borings, but perhaps three-fourths the stratum thickness below the surface would not be far from right in a majority of cases. Above this line oxidation is complete, the soil is thoroughly weathered, dark in color, crisp, firm, friable and crumbly, and is normally moist or quite dry. Below it, the color is lighter brown, the texture gummy and unyielding, and the consistency is generally that of stiff putty. Obviously, this line represents a critical elevation, since cuts that either pierce it or approach it too closely are almost certain to cause trouble. Hereafter, it will be designated and referred to as the "weather line." Above it complete weathering has occurred and conditions are stable. Below, and between it and the residual clay line, unstable conditions are found since the soil is in various transitional stages of oxidation, the latter process probably progressing in increasing degree from bottom to top.

The chemical composition of the clay in the third stage of weathering, above and below the weather line, is shown in Table 1. Practically no difference can be detected, chemically, between the stable and unstable conditions, but the colloidal content of the unweathered portion is doubtless very high.

The entire depth of the loessal deposit as well as the thickness of layers in any one of the three stages of weathering can naturally be expected to vary considerably in different localities. However, 12 feet can be said to be the most common depth with the intermediate layers spaced proportionately. On this basis, a typical section (fig. 2) may be described as follows: Total thickness of deposit, 12 feet; thickness of first stage, 4 feet; thickness of second stage, 4 feet; thickness of third stage, 4 feet. The weather line in this case would be three-fourths the thickness of the third stage, or 3 feet below the ground surface.

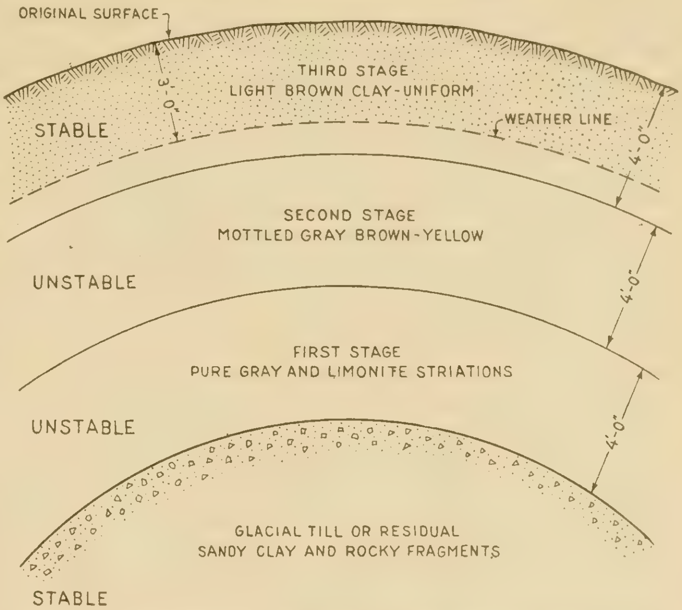

FIG. 2.—Typical diagram of three degrees of weathering of the loessal clays of Northeastern Iowa

TABLE 1.—*Chemical compositions of soil in third stage of weathering*

Substance	Chemical composition of stable material above weather line		Chemical composition of unstable material below weather line	
	Air-dry basis	Dry basis	Air-dry basis	Dry basis
	Per cent	*Per cent*	*Per cent*	*Per cent*
Silica (SiO_2)	71.10	76.41	71.80	76.28
Iron and aluminum oxides (Fe_2O_3, Al_2O_3)	17.16	18.44	16.41	17.44
Calcium oxide (CaO)	1.31	1.41	1.44	1.53
Magnesium oxide (MgO)	1.30	1.40	1.32	1.40
Alkalies and undetermined	2.18	2.34	3.15	3.35
Loss on ignition	6.95	----------	5.88	----------
Total	100.00	100.00	100.00	100.00

All the foregoing discussion has been confined to soils in their original or natural state. The situation existing in failures on graded roads can not be so simply defined. Here, the normal processes of oxidation have been interfered with and the three degrees of weathering are not so clearly apparent. If the cut is deep enough to remove the third stage and sufficient time has elapsed to allow partial oxidation of the first stage, only soils in the second stage of weathering may be present. In time, these soils would no doubt gradually become further oxidized and finally pass into a stable condition, but it would be futile to hazard a guess as to the number of years required. The churning and kneading action of traffic also complicates the classification by producing an unnatural mixture at the road surface.

As long as surplus water is kept away from the upper 2 feet of this material it forms a firm roadbed with a tough, rubbery crust. Once it becomes saturated, however, (and its affinity for capillary moisture is very great) the water clings tenaciously and whatever structure it may have possessed is immediately broken up. In this condition it is extremely soft, sticky and plastic, and shifts about readily under traffic. If enough water is present it can be made to flow like thick molasses.

The only time when such a condition occurs naturally is in the spring of the year when frost is going out of the ground and large quantities of water are drawn from the wet layers beneath by capillary action. At this time, downward movement of excess water is shut off by frost and escape into side ditches is prevented by frosty, plastic shoulders. If tile drains are present, the chances are they lie in a bed of the same material which has flowed over and effectually sealed the joints. About the only way in which surplus water can be removed, then, is by evaporation which, at this season, is very slow. Relief can not come until all frost is gone and the opportunity for vertical and lateral percolation is presented.

SOURCE OF WATER

Nothing was more clearly demonstrated in the investigation than that water is not delivered under pressure from underground sources, either as springs or seepage veins. The water is purely of local origin and is held permanently at considerable depths by the peculiar capillary properties of the soil in question. In only a few instances was flowing water encountered and these were in localities remote from failures and at depths that could cause no trouble.

In most of the test holes, a layer similar to soft putty was struck at a depth of 6 to 8 feet and this extended, as a rule, to 12 feet. Below this point the material seemed to become drier. The residual sandy clays were, for the most part, relatively dry. This leads to the belief that there exists permanently, somewhere between the ground surface and the residual subsurface, a layer of saturated material which dries out slowly by percolation from below and evaporation from above. Local rains, of course, continually replenish the moisture removed in this way. The only effect of a long, hot dry summer would be to reduce the thickness of the saturated layer.

In a few places, the roots of large trees had extracted nearly all the surplus water and wherever this had occurred, borings showed the soil to be quite dry or merely damp all the way down. Other places were noted, however, where the effect of trees was not so pronounced. Strange as it may seem, the soil, in midsummer is noticeably drier in the failures than in any other part of the road. In all probability this is due to the fact that the loessal blanket is usually of minimum thickness in failures, causing a corresponding thinness in the saturated layer, which in turn produces relatively dry soil conditions because less water is held in storage.

A good idea of the changes in moisture content that occur between April and July may be had from Table 2, which gives comparative data on material taken from the same points and same depths during the months named.

MANNER OF OCCURRENCE

From the foregoing discussion it is plain that whenever cuts of more than 2 to 4 feet below the ground surface are made, the danger zone is generally entered and a failure is likely to occur at the crest of the puncture. (Case I, fig. 3.) If the cut is deep enough entirely to remove the loessal deposit and intersect the residual subsurface, failures commonly take place on either one or both sides of the crest. (Case II, fig. 3.) In this connection, the crest of the puncture may or may not coincide with the high point of the road grade line.

Where the road lies in a steep side-hill cut, the stable material from the high side has generally been deposited in the outer half of the roadbed and soft spots normally develop in the inner half only. (Case III, fig. 3.) Occasionally a freak formation is encountered where deep cuts, penetrating the residual subsurface, do not result in a failure on either side. (Case IV, fig. 3.)

In other localities the improved roadbed was observed to be 3 feet or more below the ground surface with no failure in evidence. Such a case could usually be explained by the fact that a fill or only a very light cut had been made on the old roadbed which had previously been eroded well below the original surface. The weather line, under these circumstances, had had sufficient time to penetrate below the grade of the old road. (Case V, fig. 3.)

TABLE 2.—*Comparative moisture properties of material taken from the same points and depths within and beyond areas of failure on the Elkader-McGregor Road in April and July*

Location of test hole	Depth of strata	Depth, July sample	Description	Consistency	Capillary capacity	Moisture equivalent		Moisture content	
						April tests	July tests	April, 1925	July, 1925
	Feet	*Feet*			*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Station 556+75. Within failure.	0.5 to 4.8	2.8	Mottled gray and brown, well mixed (second stage)	Soft putty	33.8 34.5	22.1	27.6 27.6	25.9	26.4
	4.8 to 9.1	4.8	Pure gray with limonite (first stage)	do	33.7	24.8	24.8 23.8	25.4	25.0
	9.1 to 13.4	9.8	Yellow-brown sandy clay (residual)	Stiff put	28.0	18.0	23.4 20.8	18.5	21.5
	13.4 to 16	13.8	Same—slight changes in texture	Forms ball	26.2	20.7	22.2 19.2 20.8	18.3	18.2
Station 553+00. Beyond failure.	4.9 to 6.1	5.8	Light brown clay (third stage below weather line)	Soft putty	38.2	31.6	25.1 26.7 25.0	29.3	30.2
	6.1 to 8.9	7.5	Mottled gray and brown (second stage)	do	34.5	32.2	27.4 27.8	26.2	25.7
	8.9 to 17.9	10.1	Pure gray with limonite striations (first stage)	do	33.8	22.8	27.8 27.4	24.2	24.3
	17.9 to 18		Sandy blue clay (residual)	Stiff putty		16.8		17.0	

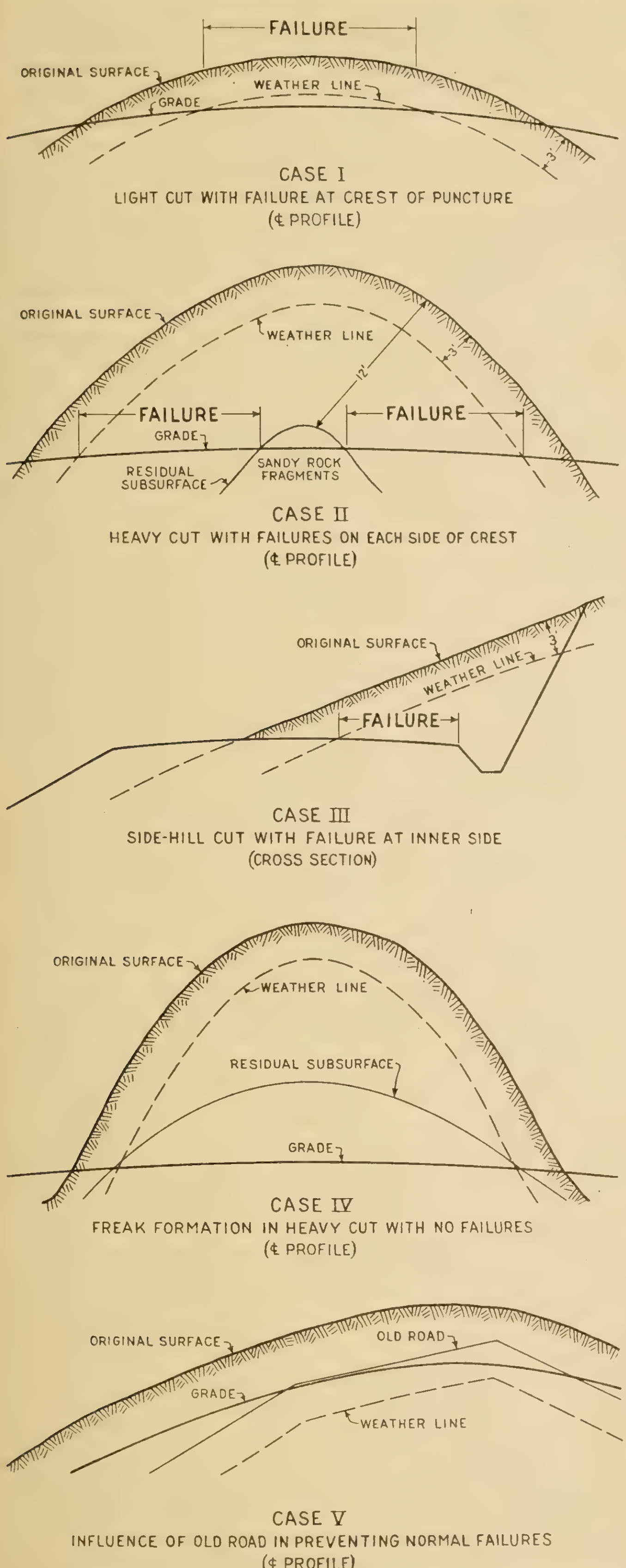

FIG. 3.—Classes of cuts in Clayton County roads and their effect upon the location of failure

PROPOSED TREATMENT

For improved roads.—As remedies for existing failures, many ideas have been advanced and these will be touched upon later, but it seems to the writer that the simplest, cheapest, and most practical method would be to duplicate within the failures conditions that have been found to exist outside, beyond either extremity. This means that the unstable material now present must be entirely removed to a depth of at least 2 feet and well-weathered topsoil or "black dirt" used to replace it. Every facility should be provided to allow the water from melting frost to escape to the side ditches without puddling at the surface.

For this purpose, the writer believes the excavation should extend entirely across the roadbed, should be given a slight crown, and should be paved with a thin layer of tough sod or grass roots (no long grass or weeds) before backfilling with topsoil. The sod would then act as an insulating layer separating the stable and unstable material and would also provide a permeable mat through which the water that rises from below could seep into the side ditches and flow away. Any organic matter like sod will naturally rot in time, but if air is excluded and the sod is kept moist, it is believed that its decomposition will proceed very slowly. Even after it has rotted out, many minute root cavities willl remain and provide a permeable passageway for seepage. Such use of sod can be defended on the ground that it is always removed from the base of earth dikes and dams for the reason that it does permit easy percolation of water.

The side ditches should of course be deep enough and have sufficient fall to assure the removal of seepage water as rapidly as it arrives. Any rain that falls during the critical period will be disposed of in the same way as in other parts of the road. Care should be taken to see that the topsoil extends well beyond the limits of the failure and that it feathers out into the stable portions of the road. One other point that commends itself in this regard is the fact that all materials necessary for the treatment are available at the site where needed. The essential features of the treatment by this method are shown graphically in Figure 4.

Two of the worst failures in the entire section were treated in this manner during the fall of 1925, and an inspection made during April showed the treated sections to be in good condition, although soft spots have developed as usual in other cuts and in the locations predicted.

For unimproved roads.—For roads not yet improved in this section, two methods of procedure suggest themselves. Either (1) the grade line can be adjusted to avoid cutting the hills by making heavy fills with borrowed earth, or (2) cuts and fills can be balanced in the usual way by making allowance for the extra depths of excavation in cuts necessary to provide for replacement with weathered material. The writer is inclined to favor the second method since construction can be carried on in the customary way and the treatment can be provided at small additional cost; but some situations no doubt exist where the first method would be preferable. In the long run, of course, topographical conditions would determine which is the most feasible in any given case. The first method, however, does possess a real advantage in that it is not dependent on experimental support to insure its success.

OTHER PROPOSALS

Rock treatment.—Figure 5 illustrates the method of rock treatment, suggested by Mr. Hutton, that has been tried with considerable success in a number of failures. The chief objection to its use lies in the expense of application and the fear of some engineers that it will prove to be temporary, since water has

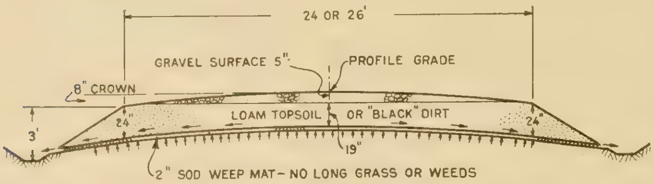

FIG. 4.—The essential features of the top soil or black dirt treatment. Any existing tile lines should be excavated and relaid in black dirt

been observed oozing up between the rocks. This or any other method would doubtless be more effective and traffic conditions would be greatly improved if the unstable shoulders were entirely removed.

Sand treatment.—The method of replacing the unstable material in failures with sand undoubtedly possesses considerable merit. Sand is heavier than "black dirt" and may make a firmer roadbed without hindering the passage of water. Its grains lack cohesion, however, and it would probably work its way down into the saturated clay and disappear (as gravel does at present) unless separated from the clay by a layer of boards or other impenetrable material. In exceptional places, where the grade line lies only a foot or so above the residual subsurface, sand dumped into the failure would effect a cure.

Lime.—The addition of large quantities of lime, well mixed with the clay, would be of considerable benefit in breaking up its dense and gummy structure. Experiments may show that this process progresses to a sufficient extent to cause relief. It is common practice among farmers to use lime for this purpose.

Use of tile.—There is little doubt that the benefits from tile drains as ordinarily laid in this material do not justify their cost. Even if their joints are not sealed by the plastic clay, the tile lie at such a depth as to be below the thaw line for several weeks and hence are rendered inoperative at the most critical time. After the frost has disappeared, however, they certainly would have some effect (with open joints) in reducing the amount of water present at the time of freezing in the fall.

The writer can not help but believe that a line of tile under either one or both shoulders, laid in a thick bed of "black dirt," with spurs angling into the failures at frequent intervals, might effect a cure. If used in conjunction with other methods, such tile would surely serve to produce less aggravated conditions in the spring.

Trees.—All trees, and especially some varieties such as willows, have a well-known capacity to absorb large quantities of water during the growing season. Wherever a number of trees were found during the investigation close to the right of way, their effect on the moisture content of the roadbed was noticeable, and some stretches of good road can be cited that would probably have been failures without the protecting presence of trees. Where large trees exist at the site of cuts, their presence constitutes a fortunate coincidence. To plant them, however, and then await their slow development, can hardly be seriously considered as a measure of practical relief.

Burning.—The writer has been informed on good authority that some railroad companies, owning mileage in similar soil conditions, make a practice of burning the clay to hasten oxidation and destroy its unstable properties. Briefly, the process is said to consist in stripping off the top layer, applying a hot flame to the subgrade and then replacing the surface soil in its original position. This kind of treatment would necessitate a large plant investment, would require skilled labor to operate it, and would be expensive to maintain. It might be tried in case the less expensive and more practicable methods failed to give relief.

Short sections of pavement.—It is said to be customary in Wisconsin to cure isolated soft spots (presumably of the same nature as those in northeastern Iowa) by constructing short sections of concrete pavement to distribute traffic loads over wider areas and thus prevent failure. This practice has many features to commend it if the necessary expense can be met. Some doubt would seem to exist, however, as to whether a permanent pavement could be easily maintained with such unstable material directly beneath.

Temporary expedients.—In order to avoid closing the roads altogether for several weeks in the spring, it is common practice to bridge over the failures with heavy planks, laid directly on the yielding surface. The planks are later removed and piled at some convenient point for use the following spring. This practice requires no comment other than that it can not be condoned on any but emergency grounds.

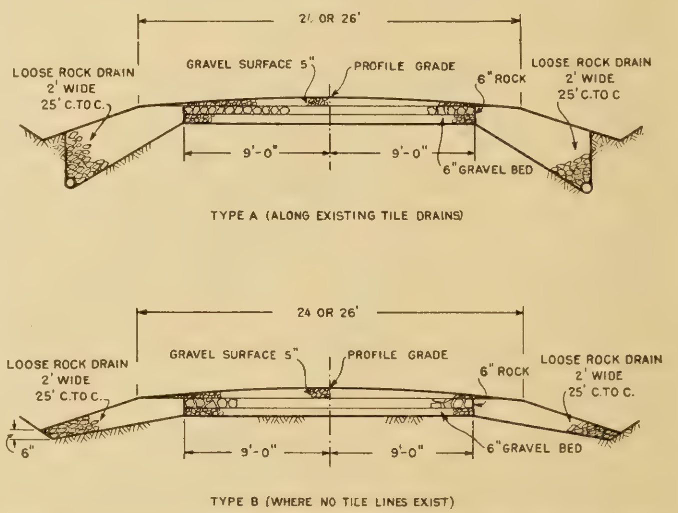

FIG. 5.—Rock and gravel surface treatments shown on the plans of the Iowa State Highway Commission

Another temporary expedient, has been to corduroy the wet spots with logs and long timbers placed transversely across the road. This primitive method is of historical interest since it has been used from time immemorial to bridge any and all kinds of mud holes, and, where logs can be kept permanently wet, it is entitled to some consideration. At least it can boast the merit of cheapness.

(Continued on page 66)

THE VALUE OF THE FOREMAN ON FRESNO AND WHEEL SCRAPER WORK

Reported by ANDREW P. ANDERSON, Highway Engineer, Bureau of Public Roads

THE value of the foreman in road grading work with fresnoes and wheel scrapers is well illustrated by data recently obtained on two jobs studied by the division of control of the Bureau of Public Roads.

The first study was a fresno job on which, at first, there was practically no effective supervision, although the work was nominally in charge of a very inefficient foreman. Later this same outfit, while operating on the same job and under practically identical conditions, was placed under the supervision of a foreman who effectively devoted his entire time to the work. The difference in the rate of operation at once became apparent and is clearly shown by the graphs in Figure 1.

Before the new foreman took hold of the job, the time taken for the performance of the several operations involved in a round trip, other than the direct haul and return amounted to 1.84 minutes, or 110 seconds, and the teams were driven at an average rate of only 179 feet per minute—a trifle more than 2 miles per hour—which is abnormally slow. When the foreman took charge, however, the time constant went down to 1.12 minutes or 67.2 seconds, while the average speed of the teams increased to 217 feet per minute or almost 2½ miles per hour. The time of performing the operations of loading, turning, and dumping, including all waits was, therefore, reduced from 1.84 minutes to 1.12 minutes, or 39 per cent, and the average operating speed of the teams was increased from 179 feet per minute to 217 feet, or 21 per cent.

The effect on the size of the average load carried to the dump was also very marked. Before the new foreman arrived the average load carried to the dump was 0.23 cubic yard. After his arrival the average load increased to 0.28 cubic yard, or over 21 per cent. For a haul of 100 feet the output of the outfit was thus increased about 83 per cent, while for a 400-foot haul, the corresponding increase in output amounted to about 64 per cent. The comparative effect of the foreman on the various operations is shown in more detail in Table 1, which represents the average results of two weeks of operation of the outfit under practically identical conditions except as to supervision.

TABLE 1.—*Stop-watch study of the fresno job before and after arrival of foreman*

Operation	Average time required: With foreman	Average time required: Without foreman	Difference	
	Seconds	*Seconds*	*Seconds*	*Per cent*
Loading	14.4	19.3	4.9	[1] 25
Dumping and turning	17.3	20.6	3.3	[1] 16
Turning at cut	14.0	18.8	4.8	[1] 26
Throw back	3.4	5.3	1.9	[1] 36
Waiting or idle	18.0	46.0	28.0	[1] 61
Total and average	67.1	110.0	42.9	[1] 39
Average load (cubic yards)	.28	.23	.05	[2] 21
Average speed of teams (feet per minute)	217	179	38	[2] 21

[1] Decrease. [2] Increase.

Most grading work and fresno and wheeler work in particular, consists of the consecutive performance over and over again of comparatively few and relatively simple operations. Consequently even very slight increases in the average time taken to perform each or any of these repetitive operations accumulate during the course of the day to rather surprising totals, which are clearly reflected in the reduced output. It is, therefore, not necessary for a grading outfit to cease operation for even the briefest periods in order to decrease its output by as much as 25 per cent, especially on short hauls. All that is necessary is an almost unconscious and scarcely apparent slowing down of the average rate of performance sufficient to add a few seconds to the time required for each of the several operations. The inexperienced observer would probably be unable to notice any loafing and even the workmen may honestly believe that they are just as busy and working just as hard in one case as the other. Only a stop-watch analysis or a check of the yardage moved will fully demonstrate the difference between operation directed with forethought and precision and the merely aimless hurry of undirected operation.

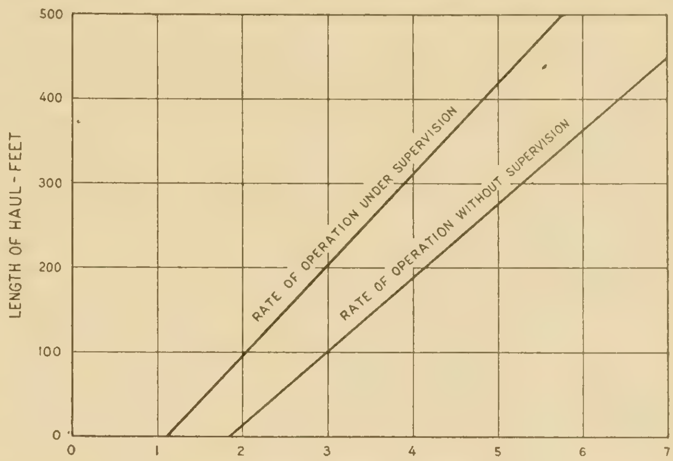

FIG. 1.—Effect of supervision on rate of operation of the fresno outfit

From Table 1 it will be seen that in this case the slowing down was of a rather aggravated form, and extended to every one of the operations, including even the speed of the teams and the size of the loads. Generally, especially during short periods of nonsupervision, the speed of the teams and the size of the load carried to the dump will be affected but little.

WHAT THE WHEELER STUDY SHOWED

The wheeler outfit presented a somewhat different set of conditions. The stock was good, the equipment first class, and the men well trained. Ordinarily the supervision was excellent and the outfit was operating at a rate well above the average. For some reason, however, it became necessary for the foreman to be absent for a period. The effect on the rate of operation was immediate and striking, as shown by Figure 2.

Under the supervision of the foreman the time constant for loading, turning, and dumping and all necessary waits was only 1.81 minutes. During his absence this time increased at once to 2.65 minutes, an increase of 46 per cent. But the average speed of the teams did not change appreciably. This was probably due, in part, to the fact that in wheeler work the pull is very light during the main hauling operation where the grade is good, as in this case, and in part to the fact that teams when in good condition and not overworked tend

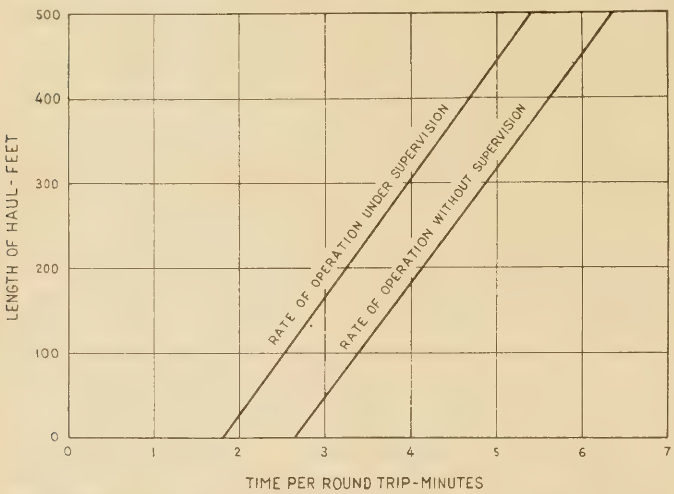

FIG. 2.—Effect of supervision on rate of operation of the wheel scraper outfit

to maintain a fairly even pace. Furthermore, the morale of the outfit was good, so there was no conscious intent to slow up the pace or decrease production. This is further reflected in the fact that no decrease was noticeable in the average size of the load carried to the dump as determined from a count of their number and a careful cross-sectioning of the cut.

In this case it was apparent that the slowing up was entirely unconscious; and it was manifested only in the slightly greater time required for each of the repetitive operations of loading, turning, dumping, etc., which, especially on short hauls, consume such a surprisingly large part of the working day. On a 100-foot haul the output of the outfit was decreased 34 per cent, but on a 500-foot haul, since the speed of the teams remained constant, the decrease in output, caused by the slowing up of the loading and dumping operations, was only about 16 per cent.

The influence of the foreman may be seen more clearly, perhaps, by an examination of the output per fresno and per wheeler as found in these studies. Thus, when no foreman was present on the job, the output per fresno on a 100-foot haul was 20 trips per hour, carrying a total of 4.6 cubic yards of material to the dump. As soon as the new forman had taken charge of the work this changed to 30 trips per hour carrying 8.4 cubic yards. In other words, the simple change from a careless foreman, frequently absent, to an alert man, constantly on the job, served to increase the amount of material each fresno placed in the dump when operating on a 100-foot haul by 3.8 cubic yards per hour. If figured at only 20 cents a cubic yard the value of the increased output per 10-day hour for each fresno was practically sufficient to pay the entire wages of the foreman. When the haul was 400 feet long the difference in the hourly output of each fresno amounted to 1.36 cubic yards or 13.6 cubic yards per 10-hour day per fresno. If the value to the contractor of material placed in the dump, even on this longer haul, were no greater than on the short haul, the increased output from three fresnoes would be more than sufficient to pay the foreman's wages. And since five fresnoes were usually on the job it can readily be seen that a handsome profit still remained for the contractor by virtue of the increased output the foreman brought to the job.

On the wheeler job the mere temporary absence of the foreman caused the output per wheeler to shrink from 8 to 6 cubic yards per hour on a 100-foot haul and from 3.7 cubic yards to 3.2 cubic yards per wheeler per hour on the 500-foot hauls. The foreman's absence, therefore cost the contractor 2 cubic yards per hour for each wheeler operating on the 100-foot haul and one-half cubic yard per hour for each wheeler on the 500-foot haul. Since there were usually five wheelers on the job operating on the basis of a 10-hour day it is clear that the contractor took a decided loss over and above the wages of the foreman during every hour he was absent from the job.

If these two studies are representative of average conditions there would seem to be no room for doubt that, on the ordinary grading job, a good foreman more than pays his own wages in the form of increased output and is therefore a necessary and vital part of the outfit.

(Continued from page 64)

Freak remedies.—Among the freak remedies that have been proposed may be listed, (1) chemical treatment, by which the weathering process will be completed overnight through the injection of some cheap chemical compound, and (2) the introduction of sawdust, hay, manure, or other rubbish into the failures with the hope that some magic effect will be produced that can not be clearly explained. It seems hardly necessary to add that, if any such remedies should meet with success, the good fortune will be purely accidental.

Since no theory is worth much until it has been tested by actual experience, the writer is in hearty sympathy with the plan to try a number of remedies that seem most feasible, with the view of adopting the one that proves best for general recommendation and use. He believes, however, that each experimental treatment should be carefully installed, under close supervision, so as to insure a fair trial to all.

MARYLAND TO STUDY CONCRETE CURING

Field tests to determine the relative merits of the conventional method of curing concrete with an earth covering as compared to concrete containing an admixture of calcium chloride with sodium silicate squeegeed on the surface are to be conducted by the Maryland State Roads Commission. The Bureau of Public Roads plans to have an observer present during the tests. Three sections of road each about 4,000 feet in length are to be built on the Maryland road system about 20 miles from Washington.

A unique feature of the tests is that double cylindrical molds are to be placed on the subgrade and filled and cured as a part of the pavement. Compression tests on these specimens will be made at ages of 1, 3, 7, 14, and 21 days.

COMPARISON OF TRANSVERSE AND COMPRESSIVE TESTS OF CONCRETE

By H. F. CLEMMER, Formerly Engineer of Materials, Illinois Department of Public Works

IT IS of primary importance in designing a concrete pavement to be able to predict within reasonable limits the actual strength of the completed slab. As Portland cement concrete has been used in the past principally where it has been subjected to compressive stresses, the compressive test has come to be general practice, and it has been carried over into the field of concrete pavement investigations, although it is a recognized fact that rigid type pavements are subjected to transverse stresses. This fact, together with the wide variation in the results of compressive tests on cores taken from pavements, has prompted general interest in the question as to whether the compressive test may be taken as a direct measure of the transverse strength of the payment.

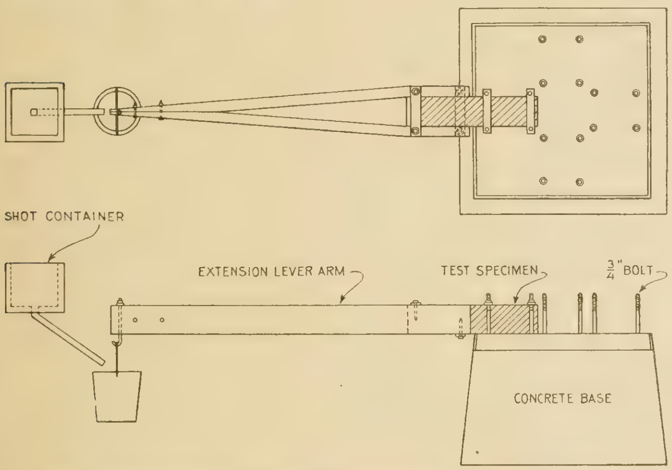

FIG. 1.—Apparatus used in making transverse tests of concrete specimens

The Illinois Department of Public Works has tested a great many cores drilled from concrete pavements and in common with the experience in other States has found a wide variation to exist in the compressive strength of the cores taken from the same job. The results have been such as to suggest that the nonuniformity may be due to the conditions surrounding the test rather than variation in the quality of the concrete. If such is the case this test indicates neither the true compressive nor the flexural strength of the slab. To throw light on this point a series of laboratory tests was conducted to determine the relation between the flexural and compressive strengths of the same concrete.

Two hundred test beams were cast, 2 from each of 100 batches and each beam was 6 by 8 by 30 inches in size. The beams were divided into three groups and one group tested at 14 days, another at 28 days and the third at 90 days. Two transverse tests were made on each beam and three compressive tests were made on cores drilled from the sections broken in the transverse test.

The transverse strengths were determined by a method developed by the Illinois laboratory which has been found to be very satisfactory. The beams are supported as cantilevers and a wooden extension arm is secured to their free ends. At its outer end the extension arm carries a bucket, as shown in Figure 1, into which shot or water is permitted to flow from another container equipped with a quick-acting valve. Uniform application of load is thus assured, and the flexural stress at the instant of breaking can be computed by taking into account the weight of the overhanging part of the specimen and that of the extension arm as well as the weight of the bucket and the shot or water it contains. The length of the specimen and the method of mounting are such as to permit more than one test to be made on each specimen; and it is particularly interesting to note that the results of tests of the same specimen rarely vary by more than a small percentage, and that exact coincidence of results is not uncommon. Figure 2 illustrates the apparatus in use.[1]

An interesting comparison of the flexural and compressive tests of identical specimens is afforded by Table 1 in which are listed the results of the two kinds of tests on 15 specimens chosen at random from the 200 beams tested. For each specimen the table shows the results of two transverse and two compressive tests and the differences between them expressed in pounds per square inch and as percentages of the minimum strength observed for each beam.

TABLE 1.—*Comparison of transverse and compressive strength of concrete specimens*

Transverse strength			Compressive strength		
Breaking strength	Difference in strength		Breaking strength	Difference in strength	
Pounds per square inch	*Pounds per square inch*	*Per cent*	*Pounds per square inch*	*Pounds per square inch*	*Per cent*
400 398	2	0.5	1,008 1,810	802	79.6
430 447	17	0.4	2,000 1,280	720	56.2
387 376	11	0.3	1,352 2,165	813	60.1
415 439	24	5.8	3,520 2,140	1,380	64.5
467 479	12	2.6	3,700 2,100	1,600	76.2
564 551	13	2.4	2,610 1,770	840	47.9
389 353	36	10.2	2,530 1,065	1,465	138.5
518 529	11	2.0	2,520 1,555	965	62.1
479 465	14	0.3	2,760 1,610	950	52.6
441 456	15	3.4	2,900 1,400	1,410	94.6
438 416	20	5.0	3,040 1,480	1,560	105.4
559 552	7	1.3	2,150 3,460	1,310	60.9
530 526	4	0.8	2,005 2,940	935	46.6
518 455	23	5.0	3,400 1,720	1,680	97.7
614 573	41	7.2	4,150 3,040	1,110	36.5
Average		3.7	Average		72

The wide variation between the compressive strengths observed for the same specimen is typical of the difference observed in tests of cores drilled from the same sections of concrete pavement. That no such difference exists in the actual strength of the concrete is clearly indicated by the remarkable con-

[1] This apparatus is now being used by a number of other laboratories including that of the Bureau of Public Roads.

sistency of the transverse tests. The location and distribution of the coarse aggregate within the core as well as the nature and size of the coarse aggregate underlying the surface doubtless affect the compressive test results; and the different moduli of elasticity of the coarse aggregate and the surrounding mortar must also be considered, especially when the cores are tested in a universal testing machine which applies the load at a nonuniform rate.

Fig. 2.—Apparatus used in determining transverse breaking strength of concrete beams

Analysis of the results of these tests suggests several questions, among which the following seem to be of sufficient importance to warrant further investigation: (1) Does the nature and strength of the outer layer or tensile fiber control the break in the transverse test or does the whole cross section at the plane of failure control it? (2) To what extent does the relative moisture content in cylinders and beams affect the respective test results? (3) To what degree does the rate of application of the load affect the test results? (4) Does the drilling of cores with the Calyx core drill cause any structural damage to the resulting core that reveals itself in the compression test?

Mainly to provide the answers to these questions the committee on tests and investigation of the American Association of State Highway Officials has planned a series of tests to be assigned to various cooperating agencies including the Illinois highway laboratory. The tests contemplated are as follows:

1. A series of tests on drilled and cast cores of concrete in which bearing areas have been carefully prepared by the arrangement of a given number of pieces of coarse aggregate. This test is suggested by G. W. Hutchinson, former testing engineer of the North Carolina Highway Commission. It has the object of determining the effect of the distribution of aggregate on compressive strength. Tests will be varied to include many combinations of aggregates. Age variation will not be an important factor.
2. Tests to determine the effect of drilling on the strength of cores.
3. Tests to determine the distribution of fiber stress in concrete beams. This may be accomplished by testing specially constructed beams having monolithic built-up layers of various types and thicknesses of concrete.
4. Development of a compression test in which uniform application of load is obtained.
5. A series of tests to determine the effect of moisture content on both compressive and transverse strength of concrete.

SLABS FOR DELAWARE RIVER BRIDGE TESTED

During the last four months the Bureau of Public Roads in cooperation with the Delaware River Bridge Joint Commission has conducted tests on concrete slabs similar in design to those used on the Delaware River bridge but smaller in size. This bridge now nearing completion is the longest suspension bridge yet constructed. The span between towers is 1,750 feet and the bridge is carried by two 30-inch cables.

The floor system consists of 6-inch concrete slabs of 1 : 1½ : 3 mix with a 2½-inch asphaltic wearing surface. The slabs are 57 feet long, by 41 feet wide and are supported by girders spaced 3 feet, 10 inches and running parallel to the long axis of the slab. The slabs are reinforced with fabricated trusses 4½ inches deep, spaced 6 inches on centers and running normal to the supporting girders and also with ½-inch round, deformed bars spaced 6 inches in the bottom and 12 inches in the top, both sets running parallel to the girders.

To check the adequacy of this design a slab similar in design and method of support but with only one-sixth the area and without the asphaltic surface was constructed at the Arlington experimental station of the bureau. Materials were the same as those used in the bridge floor. Test cylinders of the concrete showed a compressive strength of 5,000 pounds per square inch.

The first test consisted of applying a static load at the center of the slab in increments of 7,500 pounds up to a total of 30,000 pounds. Stresses in the top and bottom of the slab along both axes were measured with graphic strain-gauges. These gauges were also used to measure the stresses in the reinforcing trusses directly beneath the load. Deflection of the slab was measured at various points. As a result of this test, the entire slab was found to act as a simple plate, deflection being practically symmetrical about both axes. The maximum stress found in the trusses was 11,000 pounds per square inch. Maximum compression in the concrete was 580 pounds per square inch along the axis normal to the supporting beams and 1,200 pounds per square inch along the other axis. The deflection under the 30,000 pound load was 0.1 inch and no permanent set was found.

Impact tests were next made at the quarter point of the slab and centrally between supporting beams. The impact machine was adjusted to represent a wheel load of 15,000 pounds, the unsprung weight being 2,060 pounds, and this load was dropped one-half inch. A series of 3,000 blows was delivered, resulting in the formation of four hair cracks under the load extending outward for a distance of 14 to 18 inches. These cracks were 6 to 8 inches in length at 300 blows and showed no increase after 1,000 blows. The maximum deflection was 0.120 inch and no permanent set was measurable.

The impact machine was then moved to a new point and adjusted to give a drop of one inch without change in loading. The impact pressure developed was 18,540 pounds which is approximately equivalent to that of a 7½-ton truck with maximum overloading. After 3,000 blows no change was noted other than the formation of six hair cracks, under the load, 16 to 18 inches long, although the slab was loosened on the supporting beams and moved diagonally about two inches.

From these tests it is concluded that the design is sufficient for any loading likely to come upon the bridge. Additional tests are now being made to secure information for use in designing this type of slab.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
No. 1259. Standard Specifications for Steel Highway Bridges; adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94. TNT as a Blasting Explosive.
331. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60. Federal Legislation Providing for Federal Aid in Highway Construction.

FARMERS' BULLETINS

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*505. Benefits of Improved Roads. 5c.

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*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.
914. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

APRIL 30, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 11,167,362.27	$ 5,357,803.24	622.3	$ 5,846,496.88	$ 2,768,529.70	263.7	$ 133,112.16	$ 66,556.08	0.8	$ 3,293,368.12	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	1,230,100.53	748,233.76	102.3	1,067,930.55	715,949.79	78.8	537,790.19	369,162.88	28.4	2,767,782.63	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	4,414,693.51	1,972,083.73	237.0	5,176,455.22	2,488,384.49	331.5	536,670.32	236,160.00	37.4	1,528,994.06	Arkansas
California	22,346,175.99	10,719,249.61	894.8	4,135,497.87	1,895,395.85	138.8	11,389,368.70	5,659,451.89	325.2				3,798,717.65	California
Colorado	11,876,703.94	6,067,814.34	651.2	1,186,883.79	615,357.01	55.6	4,530,171.83	2,314,740.49	239.7	583,307.79	276,461.34	20.7	3,051,438.82	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	855,927.90	281,217.14	15.5	1,788,444.35	559,254.68	27.1	1,053,917.89	309,237.47	14.3	1,364,603.05	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1	636,492.48	285,474.95	17.2	791,364.06	315,854.75	17.8	905,863.78	196,263.44	22.7	180,274.21	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				10,476,626.23	4,978,155.14	309.2				1,701,310.89	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	4,087,584.44	1,989,360.88	243.8	11,293,499.44	5,572,699.50	692.0	2,190,368.48	1,069,889.53	60.8	393,636.63	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	1,471,405.96	923,019.03	113.8	2,369,733.91	1,484,469.49	121.1	1,094,504.27	624,863.37	84.5	711,942.85	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	3,367,972.82	1,623,785.54	113.1	5,990,923.37	2,892,045.86	204.5				6,676,290.32	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	2,537,704.28	1,229,628.96	87.5	16,674,122.40	7,796,493.09	450.3	527,415.47	261,266.45	13.2	2,354,510.82	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	722,462.52	320,365.06	36.5	9,087,872.52	4,103,451.89	567.1	3,367,948.70	1,649,290.69	256.7	2,304,962.37	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	6,381,456.99	2,812,491.49	324.4	9,989,632.53	4,022,925.80	536.9	2,213,880.93	709,606.67	147.0	2,164,113.72	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	4,492,826.73	1,670,347.39	116.6	6,600,186.13	3,122,725.52	322.7	319,991.08	143,098.54	3.0	2,070,642.96	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,740,934.40	792,419.06	114.7	3,447,327.14	1,673,255.75	162.9				1,526,862.33	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	573,271.45	284,637.06	22.2	1,809,495.54	706,814.24	57.5	92,205.65	46,102.82	4.7	1,519,403.55	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,792,436.20	1,263,608.07	128.9	169,995.00	84,997.50	10.8	1,408,591.11	661,317.24	70.9	65,751.04	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	3,618,599.61	1,005,264.48	61.8	3,486,954.51	957,291.35	48.2	505,268.22	146,144.20	7.6	2,532,364.69	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,613,563.66	4,437,617.17	346.5	9,009,665.64	4,059,552.31	219.1	191,176.00	83,048.13	5.3	4,433,830.48	Michigan
Minnesota	30,415,685.89	12,738,642.04	2,721.2	5,271,152.99	2,284,974.52	397.0	8,504,674.84	2,660,600.00	556.3	1,399,085.94	276,900.00	60.0	1,630,663.44	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	3,045,890.27	1,521,945.91	226.4	8,025,201.25	3,981,605.94	417.2	1,344,598.93	580,446.41	39.6	1,055,317.01	Mississippi
Missouri	17,368,156.57	8,219,411.43	1,118.9	9,012,250.85	4,338,296.13	325.6	21,125,049.48	8,446,115.99	592.3	1,378,318.45	610,483.74	52.9	1,172,128.71	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	1,244,383.40	1,015,942.74	133.3	1,389,599.51	936,734.74	107.3	945,723.12	513,821.09	110.2	5,640,863.28	Montana
Nebraska	9,306,374.36	4,389,523.50	1,570.6	1,968,824.26	956,092.64	157.3	11,339,124.08	5,616,483.49	1,144.9	2,073,991.43	1,032,612.40	207.5	2,640,522.97	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	2,207,013.84	1,727,392.04	167.8	3,577,844.16	3,026,915.07	342.2				952,608.11	Nevada
New Hampshire	4,165,687.86	1,986,226.87	209.1	826,870.74	391,223.20	29.5	582,611.55	274,125.52	17.7	45,925.79	22,962.89	1.4	494,953.52	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	4,110,583.94	1,191,983.72	65.5	6,193,163.14	2,434,752.44	26.0	420,870.40	66,450.00	4.4	953,553.85	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	3,568,872.54	2,326,079.86	329.6	1,456,200.02	966,613.62	116.4	74,555.37	46,757.90	5.4	2,718,864.01	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	10,721,251.06	4,261,605.31	273.5	33,121,607.58	9,548,602.24	602.8	5,589,573.00	1,244,765.00	83.3	6,761,145.92	New York
North Carolina	21,014,450.41	8,746,454.59	1,119.8	3,923,647.63	1,599,863.21	93.9	8,784,054.54	3,731,110.13	210.0	1,644,639.01	822,319.50	44.0	817,458.57	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1,917.5	1,437,900.21	739,855.63	275.6	3,572,688.81	1,768,351.71	446.7	2,596,504.84	1,434,751.65	376.0	1,536,769.54	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	6,059,391.44	2,107,214.63	171.9	8,531,417.31	3,394,848.69	272.2	4,151,653.81	1,383,537.13	108.9	3,601,201.62	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	5,514,576.70	2,561,712.80	225.5	4,580,792.94	2,182,243.50	190.7	634,804.52	307,281.98	31.3	1,335,658.38	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,325,783.62	1,256,645.21	111.3	2,290,319.28	1,378,618.00	119.0	1,484,192.23	714,382.05	55.7	387,337.11	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	15,769,432.05	4,561,888.14	286.9	27,050,703.72	7,588,139.38	546.2	2,862,755.50	930,606.84	61.5	2,036,122.67	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,360,119.89	439,140.97	21.9	1,531,802.80	427,155.00	28.5	45,705.55	13,050.00	0.9	668,534.94	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.9	3,115,050.25	1,313,931.11	159.4	6,394,991.78	2,856,526.84	287.3	663,763.43	306,165.95	51.3	203,632.56	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1,447.9	4,727,532.26	2,292,649.22	649.0	3,241,938.78	1,538,501.55	501.9	89,856.30	44,928.15	18.0	1,300,832.08	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	7,522,750.94	3,388,679.66	259.4	6,901,688.34	3,220,489.13	251.7	1,319,807.48	539,903.05	19.4	1,399,439.39	Tennessee
Texas	54,120,970.83	21,057,940.12	3,907.1	13,536,268.72	5,754,767.45	827.9	18,908,193.36	8,449,040.77	1,048.8	4,012,119.35	1,794,109.09	113.1	3,550,573.57	Texas
Utah	6,259,159.41	3,818,836.91	423.1	1,211,294.46	764,474.35	63.1	2,488,054.96	1,789,788.36	214.3				1,445,679.38	Utah
Vermont	3,015,174.51	1,452,884.45	107.8	1,226,868.13	564,805.06	26.7	1,024,826.45	423,905.57	23.2	483,741.14	169,091.25	5.7	657,810.67	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,316,348.84	3,848,702.99	303.5	6,929,855.37	3,031,274.42	209.0	592,816.29	288,497.54	17.8	61,040.85	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,591,201.52	1,607,897.59	137.2	2,525,208.65	1,232,400.00	27.6	793,644.52	424,000.00	10.2	764,266.54	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	1,639,091.90	673,572.94	57.1	5,593,959.67	2,223,538.47	141.3	1,015,249.04	427,341.73	42.0	797,764.53	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1,451.7	2,994,612.24	1,446,735.85	137.4	4,194,026.92	2,096,298.69	204.3	2,088,894.83	1,028,337.60	110.0	3,947,802.34	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	2,049,569.09	1,258,188.38	150.8	2,421,548.69	1,525,794.02	160.8	184,915.70	118,584.00	15.0	924,610.93	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	193,325,711.19	85,704,371.13	8,962.5	334,328,211.86	143,340,251.69	13,808.6	53,599,718.00	21,990,555.69	2,423.5	94,685,475.49	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 71,916,541.14 Federal aid $ 32,193,281.99 Miles 2,957.6