U.S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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VIEW ON COLUMBIA RIVER HIGHWAY, SHOWING ATTRACTIVE AND SUBSTANTIAL GUARD RAIL

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U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

PUBLIC ROADS

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THOMAS H. MacDONALD . . . . Chief of Bureau
P. ST. J. WILSON . . . . Chief Engineer

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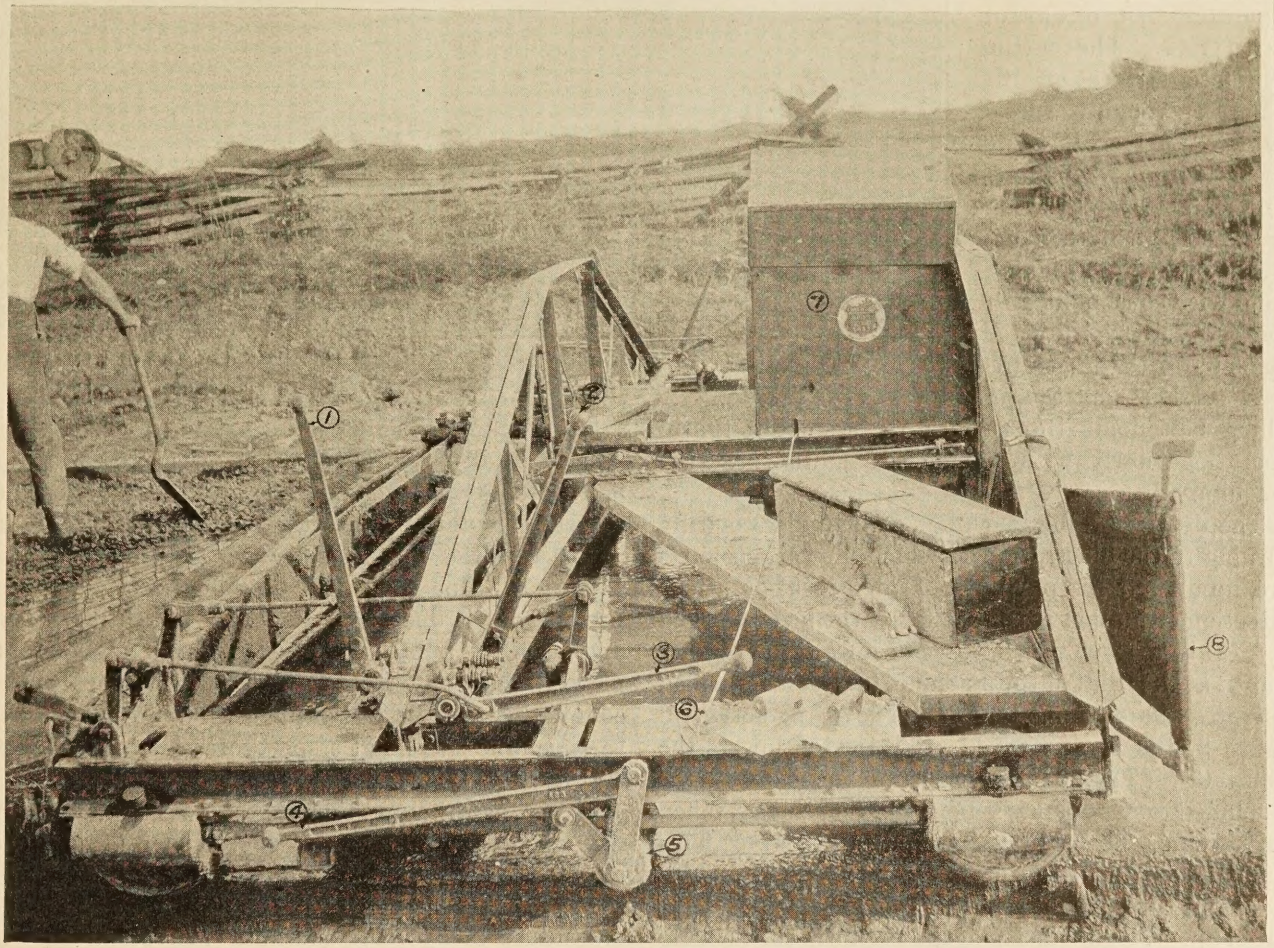
MECHANICAL CONCRETE ROAD FINISHER

By H. G. McKELVEY, Senior Highway Engineer, U. S. Bureau of Public Roads.



THE CONCRETE ROAD FINISHER. 1. FORWARD AND REVERSE LEVER. 2. TAMPING LEVER THAT OPERATES THE TAMPING BOARD SHOWN AT THE FRONT OF THE MACHINE. 3. STRIKE-OFF LEVER. 4. LEVER OPERATING ADJUSTING ROLLER, 5, BY MEANS OF WHICH ONE SIDE OF THE MACHINE IS PREVENTED FROM GETTING TOO FAR AHEAD OF THE OTHER. 6. THROTTLE LEVER LEADING FROM THE 4 H. P. AIR-COOLED NEW WAY ENGINE IN THE HOUSING CASE 7. 8. FINISHING BELT SHOWN RAISED, WHICH OPERATES AUTOMATICALLY WHEN LOWERED. THE MACHINE HAS THE SAME NUMBER OF LEVERS ON EACH SIDE AND CAN BE OPERATED FROM EITHER.

THE scarcity and high cost of labor during the past two years or more have seriously handicapped highway construction generally, but have been beneficial to the extent that they have resulted in the invention of a number of labor-saving devices which offset, to some extent, the perplexing labor problems.

One notable instance of this development is a machine which automatically strikes, consolidates, and floats concrete pavements and forms monolithic brick surfaces, with the aid of but one man for its operation for either type of pavement.

The concrete road finisher, shown in the illustration was primarily devised for concrete pavement construction, but is equally well adapted to the preparation of the concrete base for monolithic brick construction, and the tamping and shaping of the brick surface. The machine, sustained on the four flanged wheels, travels on the ordinary side forms used to confine the concrete or brick and is moved forward or backward under its own power, operating during the forward movement but usually running idle in the reverse direction. The power, for both traction and operation, is furnished by a 4-horsepower, air-cooled, gasoline engine, protected by a housing.

The three principal members of the machine operate during the manipulation of the material for a concrete pavement. They are, (1) a striking template with a metal edge, adjustable to the crown of the pavement; (2) a tamper, consisting of a heavy timber, kiln-dried and oil-soaked, and shod with a

A MIXTURE OF THE PROPER CONSISTENCY. THE MACHINE PERMITS THE USE OF DRIER MIXTURES THAN CAN BE WORKED BY HAND METHODS.

steel channel; and (3) a finishing belt attached to a supporting frame at the rear of the machine. These three parts have distinct functions to perform and operate simultaneously or separately as occasion requires. The cutting template strikes off the concrete, roughly spread by two men as it is deposited from the mixer, and shapes it to the approximate height and crown desired. The tamping device consolidates the concrete with well directed, quick blows to the required cross section; and the finishing belt floats the surface to a smooth finish, operating slowly with a sidewise motion as the machine advances.

The details of the machine can be understood by turning to the illustration, in which the various parts are numbered for purposes of identification. The direction of motion is controlled by the lever marked (1), (2) is a lever which raises or lowers the tamper shown at the front of the machine, (3) is a lever which operates the striking template, shown in action in figure 2, the lever marked (4) raises or lowers the roller (5). This device prevents one side of the machine from advancing beyond the other. When this occurs, the roller is lowered into contact with the form, lifting the wheels from the form on one side, and thus permitting those on the other side to catch up. The operation of the engine in the housing (7) is controlled by the throttle lever (6); and (8) is the finishing belt in the raised position. Released for action the belt is lowered and operates automatically with a sidewise motion. The machine has the same number of levers on each side, and can therefore be operated from either with equal facility.

This device is manufactured for use on pavements of several widths, but is not adjustable so that the same machine can be used on pavements of different widths. The net weight is 2,600 pounds.

TAMPER STRIKES UNIFORM BLOW.

The principal benefit is derived from the action of the tamper. The device replaces the usual heavy timber tamper, which is laboriously operated by two or four men. The machine tamper strikes a uniform blow over the entire width of the pavement. By its use the consolidation is effected usually by three passages over the surface, the first time striking a long, hard blow, and the second and third times delivering short, rapid strokes. The nature of the blow is within the control of the operator.

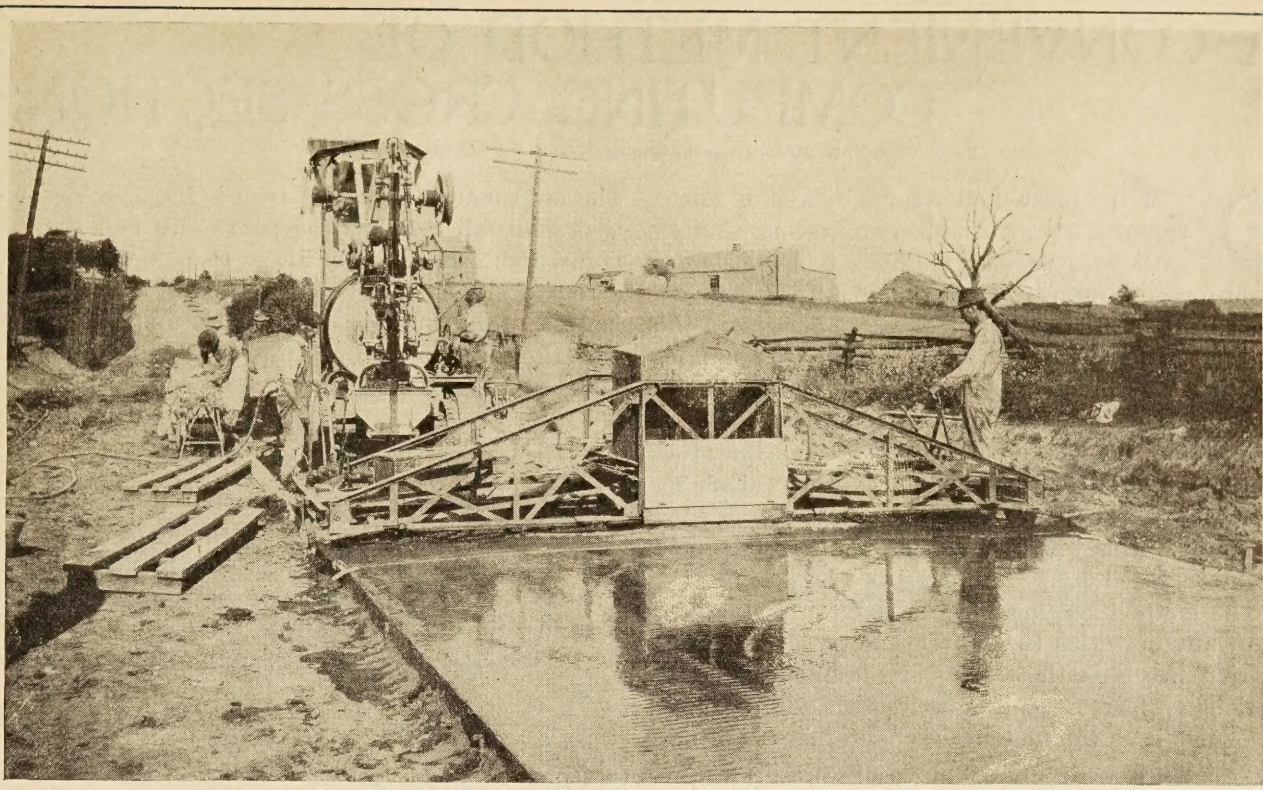
The striking member is adjustable to operate or remain idle, as occasion requires, as is also the belting contrivance. During the first and second transit over the mixture the cutting template is in place, but the belt is elevated and remains idle. The third time over the tamping device only is permitted to operate. For the fourth and last operation the tamper is adjusted to strike about 150 blows to the minute, and the belt is lowered into place, completing the work. During the forward movement the machine travels at a speed of 7 feet per minute, and in reverse it travels 30 feet per minute.

Experiments have proved that a comparatively dry mixture will produce stronger concrete than one in which an excessive amount of water is use, but the practical difficulty of consolidating and finishing dry mixtures by hand methods has stood in the way of the general employment of such mixtures. This machine permits the use of drier mixtures than could be worked by hand methods and its use should result in the production of concrete that will set quicker, form slabs of greater strength, and pavements that will be ready for service in shorter time.

ADVANTAGE IN USING DRY MIXTURES.

The machine is not effective in finishing wet concrete. It produces a wavy surface, as shown on page 5. However, this is not a defect, but rather one of the greatest advantages of the machine, for it soon becomes evident to all who are employed at the mixer that to obtain workable concrete a dry mixture must be furnished. The inspector is thus relieved of the trouble which has always been experienced in securing dry mixtures by hand methods, due to the greater effort required by such methods to manipulate the dry concrete.

On the whole this machine represents a long stride forward in the construction of concrete roads, but it is not without defects, some of which are serious.



THE MACHINE IS NOT EFFECTIVE WITH WET CONCRETE. WAVY SURFACE PRODUCED BY THE MACHINE IN FINISHING WET CONCRETE.

For example, as previously stated, each machine can be used on only one width of pavement. If, therefore, a contractor would equip himself to construct all of the widths which are specified by the different States, counties, or municipalities for which he performs work, he must have at least as many tampers as there are widths specified; and as the factory price is $1,650, such an outfit might call for a very considerable investment.

To support the great weight of the machine, forms must be given better support than they commonly are and must be rigidly secured laterally. The additional expense of this work detracts from the saving in the operations performed by the machine. But the most serious fault is that the machine can not be used in developing changing cross sections, such as are used in the superelevation and widening of curves. Sections of surface which lie on such curves must still be consolidated and finished by hand, and this fault leads to a lack of uniformity in the character of the surface of the road as a whole.

MORE BONDS.

By a vote of 800 to 63 the citizens of the Lompoc road district, in Santa Clara County, Calif., have decided in favor of a bond issue of $400,000 for the construction of improved roads.

IOWA HIGHWAYS.

Of the 99 counties in Iowa, all but 1 have adopted a definite program of highway improvement. Twelve counties have voted bond issues, the aggregate amounting to nearly $28,000,000, for permanently graded, hard-surfaced roads, which will be used with Federal-aid funds and the funds available from other sources. A great many other counties have voted for hard-surfaced roads, to be built from regular taxation.

ISSUE OF BONDS LEGAL.

The Supreme Court of Washington has decided that the plan adopted by the commissioners of Spokane County for the issuing of bonds to the amount of $3,250,000 recently voted by the people complies with the election. The point at issue was the question of the retirement of the bonds. The method adopted by the commissioners would not permit the retirement of some of the securities until about 23 years after the bonds had been legalized by the election, which was based on a 20-year issue. The attorney general had held that the method adopted did not comply with the election proposition and the State auditor had refused to pay over to the commissioners $450,000, the first issue to be sold.

A CONVENIENT METHOD OF COMPUTING CROSS SECTIONS

By G. T. McNAB, Senior Highway Engineer, U. S. Bureau of Public Roads.

ONE of the most monotonous details of a field engineer's work is the computation of cross sections. Various methods have been devised for performing this work, but all of them are open to the criticism that they are laborious in the extreme. So far as the writer knows, this applies whether they involve the calculation of the areas of cross sections by ordinary mathematical methods or whether they involve the plotting of the cross sections and the measurement of their areas by planimeter. Therefore any new method designed to simplify this work should be of interest to field engineers generally.

The method outlined below is more convenient than any of which the writer knows. As the discussion proceeds those who are familiar with the various systems of land measurement will probably recognize it as the method of computing land areas by latitudes and double meridian distances. Its value for the computation of cross sections is not generally recognized. The time required to compute cross sections by this process is remarkably short, and by the use of it the volume of work that can be done in a day is correspondingly large, so whether the method is absolutely new or not, it is worthy of much more general adoption. This method is much simpler than any other of which the writer knows and can be used in computing the area of the most complicated figures as readily as that of an ordinary cross section. One who is expert in the use of this method finds it a comparatively simple matter to compute the cross sections on as much as 5 or 6 miles of highway in a single working day. The discussion of the method follows.

Let figure 1 represent any cross section, as, for instance, one cross section of a borrow pit. Before any excavation is taken out lines of levels will, of course, be run over the area from which excavation is to be taken. Let it be assumed that the elevations on the first line run, are as follows:

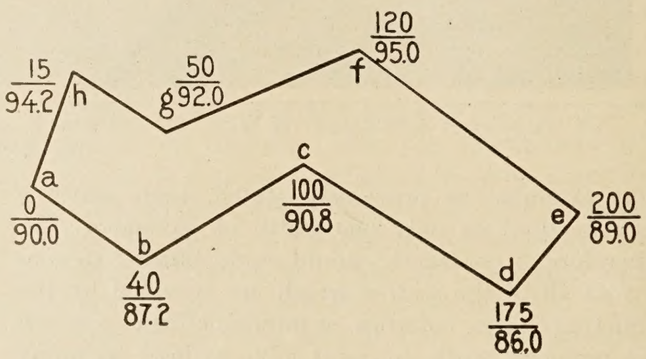
0	15	50	120	200.
90	94.2	92	95	89

Similar elevations would, of course, be taken on the other lines run. And here the first important feature of this system develops. It is a matter of entire indifference, so far as this system of computing cross sections is concerned, as to how far apart the stations are, whether they are equal distances apart or not, or as to whether there is any means provided for reoccupying any of the intermediate stations. The only matter of importance is that the excavation shall fall between the point zero and the last point occupied and that these two points shall be carefully preserved.

After excavation has proceeded for a time new elevations are taken over the course, the following notes resulting:

0	40	100	175	200
90	87.2	90.8	86	89

Again the matter of the selection of the stations between station zero and the last point occupied—in this case 200—is one of entire indifference, for the calculation of the area inclosed between the original line and the new line can be as readily calculated where the intervening stations occupied are different



(40+100)× 3.6= 504.0
(175+200)× 3.0=1,125.0
(200+120)× 6.0=1,920.0
(50+ 15)× 2.2= 143.0

14.8 3,692.0
2,005.0

2)1,687.0

843.5 Area.

(0+ 40)× 2.8= 112.0
(100+175)× 4.8=1,320.0
(120+ 50)× 3.0= 510.0
(15+ 0)× 4.2= 63.0

14.8 2,005.0

FIG. 1.—SHOWING COMPUTATION OF BORROW-PIT AREA.

from those selected when the course was run the first time, as when they are the same.

The two sets of elevations over this course form the figure,

a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*a*
0	40	100	175	200	120	50	15	0
90	87.2	90.8	86	89	95	92	94.2	90

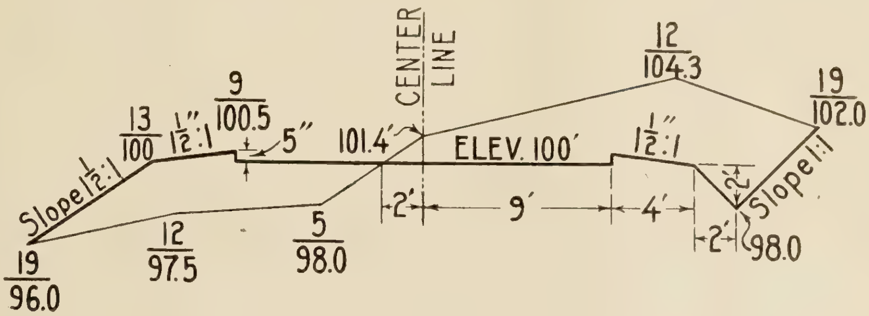
which is a closed figure. The area of this figure may be computed from any point in the series, but for ease in computation and because this is the general procedure, the zero point will be selected.

The area is computed by taking one-half of the algebraic sum of the products resulting from the multiplication of the sum of the horizontal distances from the zero point, assumed, to each end of each of

```
(0+ 9)×0.0=   0.0          (9+13)×0.5=  11.0
(9+ 9)×0.5=   9.0         (13+15)×2.0=  56.0
(15+19)×4.0=136.0          (12+ 0)×2.9=  34.8
(19+12)×2.3=  71.3          (0+ 0)×1.4=   0.0
(2+ 0)×1.4=   2.8           (0+ 2)×0.0=   0.0
                            (0+ 0)×1.4=   0.0
        ——  ————                   ——  ————
        8.2 219.1                  8.2 101.8
            101.8
            ————
          2)117.3
            ————
            58.65 Area.
```

EXCAVATION.
EMBANKMENT.

```
(0+ 2)×0.0=   0.0           (2+ 5)×2.0=14.0
(19)+13)×4.0=128.0          (5+12)×0.5=  8.5
(13)+ 9)×0.5=  11.0        (12+19)×1.5=46.5
                            (9+ 9)×0.5=  9.0
        ——  ————            (0+ 9×)0.0=  0.0
        4.5 139.0           (0+ 9×)0.0=  0.0
             78.0                  ——  ————
            ————                   4.5  78.0
          2) 61.0
            ————
             30.5 Area.
```



EXCAVATION.

```
(15+)19×4.0=136.0                   2.0
(19+)12×2.3=  71.3     (13+15)×2.0=56.0
(2+) 0×1.4=   2.8      (12+ 9)×2.9=34.8
            ————                   ————
            210.1                  92.8
             92.8
            ————
          2)117.3
           e————
            58.65 Area.
```

EMBANKMENT.

```
                2.0     (2+ 5)×2.0=14.0
(19+13)×4.0=128.0       (5+12)×0.5=  8.5
            ————       (1+19×)1.5=46.5
            130.0                  ————
             69.0                  69.0
            ————
          2) 61.0
            ————
             30.5 Area.
```

FIG. 2.—SHOWING METHOD OF COMPUTING AREAS OF EXCAVATION AND EMBANKMENT IN A SIDEHILL SECTION.

the boundary lines of the cross section by the differences in elevation of the two ends of the respective lines, considering the products derived from descending lines as negative, and those from ascending lines as positive. There, therefore, result two columns of quantities, one a negative column, the other a positive column. Thus, turning to figure 1, we have for line *ab* $(0+40)\times 2.8=112$, which is placed in the negative column because elevation 87.2 is less than elevation 90, and therefore this line is a descending line. From station 40 to station 100, which is line *bc*, we have $(40+100)\times 3.6=504$, which is placed in the positive column because this line is an ascending line. Descending from station 100 to station 175, we have $(100+175)\times 4.8=1{,}320$, which is placed in the negative column. Then from station 175 to station 200, which is an ascending line, we have in the positive column $(175+200)\times 3=1{,}125$. Returning from station 200 to station 120 we ascend, and therefore place in the positive column $(200+120)\times 6=1{,}920$. Descending to station 50 we place in the negative column $(120+50)\times 3=510$. Ascending again from station 50 to station 15 we have $(50+15)\times 2.2=143$, to place in the positive column, and finally from station 15 to station 0, the point of beginning, we have $(15+0)\times 4.2=63$, which is placed in the negative column because this is a descending line.

As these numbers are set down they are usually written in two columns as shown in figure 1. As a check on the accuracy with which the quantities have been taken off, the multipliers in both columns should be added, and if the differences in elevation have been correctly calculated the sums of the positive and of negative side will be equal. In this case the multipliers total 14.8 on both sides. Adding the products we have 3,692 on the positive side and 2,005 on the negative side, the difference being 1,687. One-half of this is 843½, which is the area of this cross section. It should be clear at a glance that the final result of this process is, as stated above, entirely independent of the number of stations appearing between station 0 and station 200, either in the original survey or in any subsequent survey. It is not necessary that the stations, other than the first and last, have any other relation to each other than that they appear on the same line. From this it should be clear that this method is of particular advantage in cross sectioning borrow pits, as it is not essential that any points be preserved in the area where work is being carried on. Indeed all levels may be taken at the most advantageous points and at those points which will tend to promote the greatest accuracy in the final result, regardless either of preceding or of successive work. This system is, therefore, peculiarly adapted to field conditions. Moreover, owing to the simplicity of the calculations any desired degree of refinement may be secured, for any increase in the number of stations occupied merely serves to increase the number of computations per cross section, but in no way increases their complexity.

Finally, the system is so simple that it can be taught to any one who can add and multiply. This is an important matter, for it enables the chief of party to train his subordinates—even the brighter rodmen and axmen—to compute cross sections quite as accurately as he can do it himself.

The application of the method to the computation of the areas of road cross sections involves processes which are similar to those which have already been described for the calculation of borrow pit areas. In this case, however, it is convenient to assume as the zero or reference point the center of the subgrade. In doing so, however, to avoid negative areas it is necessary to proceed around areas to the right of the center in an anticlockwise direction, and areas to the left in a clockwise direction. When the area is entirely in excavation or entirely in embankment, or when, on side hill sections, an area of excavation or embankment crosses the center of the road, it is necessary to consider the parts to the right and left of the center as separate areas and to bear in mind the injunction as to direction, else the correct area will not be obtained.

Figure 2 illustrates the method of computing the areas of excavation and embankment in a sidehill section. In the computations set down above the figure, it will be observed that the parts of the area of excavation on the right and left of the center, respectively, are considered as separate areas and that the algebraic signs of the partial products are determined by proceeding in the anticlockwise direction around the part to the right and in the clockwise direction around the part to the left of the center. Similarly the area of embankment which lies entirely to the left of the center is encircled in a clockwise direction.

As set down above the figure, the computations are unnecessarily extended by the inclusion of the multiplications, the products of which are zero. In practice, these can be determined by inspection and omitted. A further saving in time and labor results from the recognition of the fact that a constant road cross section yields a constant difference between the positive and negative products which result from progressing from the center reference point to the edge of the shoulder. Thus on the excavation side the products to this point are:

Plus.	Minus.
(0+9)×0=0.0	(9+13)×0.5=11.0
	11.0
(9+9)×0.5=9.0	9.0
9.0	Diffeience−2.0

On the embankment side, beginning at the shoulder and proceeding in a clockwise direction to the center, the products are:

Plus.	Minus.
(13+9)×0.5=11.0	(9+9)×0.5=9.0
11.0	(0+9)× 0=0.0
9.0	9.0
Difference+ 2.0	

The difference is 2.0 in each case, but for areas to the right of the center it has the negative sign and for areas to the left of the center the quantity is positive.

Such a constant may be determined for any given cross section, and thereafter the computation of the areas may be simplified by following the form shown under the cross section in figure 2.

DIRT ROAD EXPERIMENT.

Erie County, Pa., has 11,000 miles of highways. It has planned to construct 90 miles of paved road this year, and for the building of roads has already authorized bonds to the amount of $1,200,000, $500,000 of which were sold last year. By the end of the summer it is estimated that the limit of the county's bonding power, $1,800,000, will have been reached, and it will be necessary to submit a new bond issue to the people in November.

Because of the road situation County Commissioner W. G. Walker advocates the construction of and experimentation with dirt roads, especially for roads which can not be now paved, and has recommended that $54,000 be spent on them this year. He estimated that a well-drained county dirt road can be built at a cost of from $1,000 to $3,000 a mile.

TO INCREASE ROAD FUNDS.

As a result of recent conferences between the State road commission of Utah and commissioners from almost all the counties of the State, the commissioners of Utah county have agreed to increase the amount of taxes to be raised in that county this year for roads from $179,013.35 to $278,342.35. Other counties have under advisement increases in the amounts originally provided. The road commission finds itself in an embarrassing position from the great increase of the cost of road construction, and it is seeking ways and means for securing funds with which to prosecute the program planned.

FOR OIL ROADS.

About 40 farmers attended a good roads meeting recently held at Adair, Ill. The oil question was discussed and plans matured for about 25 miles of oil roads in one township. The funds will be furnished by the commercial association and county and township treasuries.

Put your savings into war savings stamps and Treasury savings certificates—always worth more than you paid for them and not the kind of riches that take wings.

SUBSTANTIAL AND ATTRACTIVE GUARD RAIL ON OREGON ROAD

THE GUARD RAIL ERECTED WHERE NEEDED ALONG THE COLUMBIA RIVER HIGHWAY.

NO FEATURE of our modern roadways stands out more prominently in the eyes of the wayfarer than the guard rail erected at the roadside to protect him in the dangerous places. Weakness and poor design are in no other feature more quickly detected. Flimsiness spells the defeat of its primary purpose, but to achieve mere strength of construction, without artistry in line and proportion, is to lose an excellent opportunity for attractive roadside adornment.

Several types of guard rail have been developed by the State highway departments, which combine the attributes of solidity and grace, but none is more substantial or attractive than the type developed by Multnomah County, Oreg., and adopted by the Bureau of Public Roads for western Federal-aid roads.

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This type was first used on the Columbia River Highway in places where a particularly substantial and artistic design was desired, and one that would be less expensive than the stone fence constructed along parts of the highway.

The posts, which are planted 8 feet from center to center, consist of 8-inch by 8-inch timbers, surfaced on four sides, and stand 4 feet 3 inches above the ground. Normally they are set 3 feet 9 inches into the ground, but are carried to greater depth as required by local conditions. To protect them from moisture they are painted with an approved bituminous paint for a length of 4½ feet from their lower ends.

The two rails, consisting of 3-inch by 8-inch timbers, also surfaced on four sides, are spiked or drift-bolted to the posts, and great care is observed, by drilling for bolts and even for spikes, if necessary, to prevent the splitting of the rails. They are cut to the length necessary to span two panels of the fence, and are squarely butt-jointed at the posts, the two rails breaking joints.

Before erection all contact surfaces and the tops of the posts are treated with one heavy coat of a white paint, consisting of white lead and zinc oxide in the proportions of 3 to 1 by weight, mixed with raw linseed oil and turpentine drier.

After erection the entire exposed surface of the fence is given three coats of the same white paint. As it has been found that the bituminous paint used as a preservative has a tendency to turn the white paint yellow, it has become customary to paint a black band around the posts for a distance of about 9 inches above the ground, or high enough to cover the part of the bitumen-treated section of the post which is above ground. As each post is thus painted to the same height above the ground, the contrasting color really adds to the appearance of the fence.

Bid prices for the construction of the fencing of this type on the Columbia River Highway projects ranged from 60 cents to $1 per lineal foot. The estimated cost per 8-foot panel is tabulated as follows:

75 feet b. m. lumber, at $40	$3.00
Setting and framing 1 post	1.00
Paint, 3 coats	.60
Bituminous preservative per post	.20
	4.80
Plus 20 per cent profit	.96
Estimated cost	5.76

REMAINS OF U. S. TROOPER EXCAVATED ON FEDERAL-AID ROAD

Resident Engineer J. J. McCreedy tells an interesting story in connection with the construction of Idaho Federal-aid project No. 6. This improvement consists of grading and draining a highway from the town of Whitebird, situated on the Salmon River, to the plateau 3,000 feet above, and thence to Grangeville. The country is extremely rugged. In the light of the morning sun the peaks of the hills, the folds and creases in the sides of the principal and subordinate valleys, have the appearance of a blanket thrown over the bed of the giant earth.

On September 9, 1919, workmen engaged in excavating the new roadway with a caterpillar steam shovel uncovered the remains of a United States trooper. He had died with his boots on. Nothing remained of the uniform, except the black visor of the military cap, brass buttons eroded beyond recognition, and black leather half-knee boots fairly well preserved. A brass spur fastened to the left boot recalled the rigid economy practiced by the Government during the life of the soldier. Silver-rimmed spectacles rested on the skull. The skeleton was well preserved and the teeth were in excellent condition.

In 1877 the Nez Perce Indians under Chief Joseph attempted the massacre of the settlers in Whitebird and the surrounding territory. Whitebird, situated in the Salmon River Canyon, was at that time a placer gold mining camp. A troop of United States cavalry, together with local Indian fighters, was dispatched by Gen. Howard from Fort Lapwai (12 miles east of Lewiston), under the direction of an Indian guide. This treacherous informer led the band into the canyon 2 miles north of Whitebird.

Relying on his word that no Indians were near, the entire party dismounted for a rest and loosed the saddle girths on the ponies. Immediately the Nez Perce tribesmen, ambushed in the surrounding hills, opened fire, while the members of the band made attempts, with variable success, to ride in the loose saddles on the backs of the terrified horses.

In the running fight which ensued 1 lieutenant and 32 troopers were killed and the balance escaped to Grangeville in safety. The volunteer experienced Indian fighters protected the retreat and made possible the escape of the troopers, some of whom were unaccustomed to this type of warfare.

The killed were buried temporarily by their comrades, who returned a few days after the fight. Later the remains were dug up and placed in permanent graves. Some of those covered hurriedly in the shallow pits must have been overlooked in this last burial, which would account for the skeleton which was excavated 1 mile north of the point where the troop was first surprised by the Indians. The road construction force have selected a more secluded spot in which they have placed the remains.

A 20-MILE DIRT ROAD.

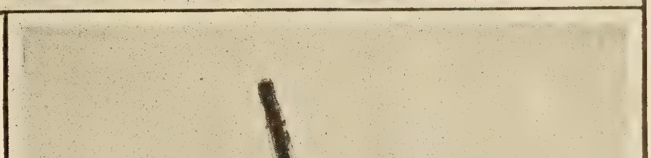
Houghton County, Mich., will this year build what is known as the Painesdale to Ontonagon County line road, which is 20 miles long and will give needed road facilities to the more populous parts of the county. Houghton County has taken over from the State the surveying of the road through Ontonagon County.

IDAHO'S 1920 FEDERAL AID.

All of Idaho's share of the Federal-aid funds for 1920, $1,159,976, is already pledged to road projects in different parts of the State, according to the State commissioner of public works.

MACHINERY REPLACES HAND LABOR ON MINNESOTA PROJECT

By GEO. C. SCALES, Senior Highway Engineer, U. S. Bureau of Public Roads.



MACHINERY ON MINNESOTA FEDERAL-AID PROJECT NO. 31. TOP—STEEL STONE AND SAND BOXES AT MIXING PLANT. SUBGRADE AND FORMS READY TO RECEIVE CONCRETE SURFACING. BOTTOM—SAND AND STONE STORAGE BINS, CRANE AND CLAM SHELL BUCKET AT RAILROAD SIDING.

DURING the past three years there has been a very considerable increase in the cost of labor per unit of production. This increase is a reflection of the higher wages which are now paid for labor and of the decreased production per man which is the usual accompaniment of a surplus of work. The present is, therefore, a time when there is special advantage in reducing the requirements for labor by the substitution of machinery.

This high cost of labor per unit of work done prevails to-day in industry generally. However, it has, perhaps, been more noticeable in highway construction than in some other activities, for the reason that highway construction is not as well standardized as are most industrial activities and that, therefore, there is not the well-established standard of individual output which careful attention to detail has made possible in these other activities. Highway construction has generally been carried on by small organizations, and the nature of the work, as well as the fact that it is not a continuous industry in most communities, has prevented the development of any large body of men who are specially trained in many of the manipulations involved in ordinary highway construction. It follows that, to the general handicap caused by the fact that highway construction is not a well-organized or standardized industry, there is to be added the fact that the industry persists in a given locality only for a comparatively short time and that the rank and file of the laborers employed are, therefore, unaccustomed to the work. From this it follows that the laborers themselves have in their own experience no ready measure of a proper day's work. Hence, there has been a very marked tendency for all phases of highway construction which involve the use of common labor to increase in price out of all proportion to the increase in the actual wages paid for the work performed. The increase in the cost of excavation is an excellent illustration of this fact. Five years ago ordinary excavation cost from 20 to 30 cents per cubic yard. The average cost was not far from 25 cents per cubic yard for ordinary light cuts and fills, which are a feature of highway construction and which involve a considerable amount of hand labor. For this work the wages of common labor were, at that time, approximately 20 cents an hour. To-day common labor can generally be secured for about 40 cents an hour, which is double the former rate: but it now is a very common thing to find bids for this same class of work running from 90 cents to well over $1 a cubic yard. The difference between the increese in the wages and the increase in the price paid for the work is the measure of the reduction in labor efficiency, due to the causes briefly outlined above.

In order to offset the falling off in labor efficiency, contractors for highway work are resorting to the extensive use of machinery. The proper performance of a machine is generally well known, and, therefore, its output is a measure of the efficiency of the operator. Thus, if a concrete mixer is designed to turn out 100 cubic yards of concrete in a given period, or if an engine is designed to haul a certain number of cars at a certain speed, or if a crane is designed to move certain loads, and if any of these machines fail to perform according to design the owner knows that, in all probability, the failure rests with the operation of the machine rather than with the machine itself, and promptly secures a new operator. In this way the performance per man can be held somewhere near to a standard schedule with more or less disregard for any general condition of labor efficiency such as exists at the present time. It is, therefore, particularly profitable at this time to use machines in the construction of highways, not only on account of the natural saving in labor, which well-designed machines make possible, but also because, during times like the present, well-operated machines tend to stimulate the productivity of labor.

MINNESOTA FEDERAL-AID PROJECT 31.

These general principles explain the extensive use of machinery by A. Guthrie & Co., of St. Paul, in the construction of Minnesota Federal-aid project No. 31. This project, involving the construction of 9 miles of concrete pavement, is divided into two sections; one extending 3 miles east from the city limits of Willmar, and the other 6 miles west from the town. The pavement is a standard one-course concrete pavement of 1:2:4 mix, and is laid 18 feet wide, 7½ inches thick at the center, and 6½ inches thick at the edges. The road was graded in 1918 so that the contract awarded to A. Guthrie & Co. involves subgrade work only to the extent of the shaping up of an existing subgrade and laying some drain tile. The main part of the contract involves the placing of 94,494 square yards of concrete pavement, for which the price bid was $2.44 per square yard.

The road is practically level, but there are occasional grades, the maximum being 2½ per cent. The Great Northern railway parallels the project for approximately a mile, and to facilitate the construction work a siding and unloading plant have been located about a mile from the east end of the project. Here the usual storage space has been provided and a construction camp established.

FEATURES OF UNLOADING PLANT.

The extensive utilization of machinery which has characterized the whole work began with the unloading plant. This consists of a stiff-leg derrick, which is provided with an exceptionally long boom. The derrick is so placed that the boom can readily drop the material taken from the cars over an extensive storage area or into a stone bin erected on the opposite side of the track. By the use of a derrick of this type it is possible to reduce the bin capacity so that the cost of loading bins is comparatively small.

The bin is of standard construction, about 40 feet long, 8 feet wide, and 12 feet deep, and was elevated far enough above the ground to provide for gravity discharge. The length of the bin is so divided that one-third of the space is reserved for sand and two-thirds for gravel; six chutes are provided, so that as the industrial cars are spotted for loading six of the buckets which they carry can be loaded at one time. In these respects the bin, as erected, is normal.

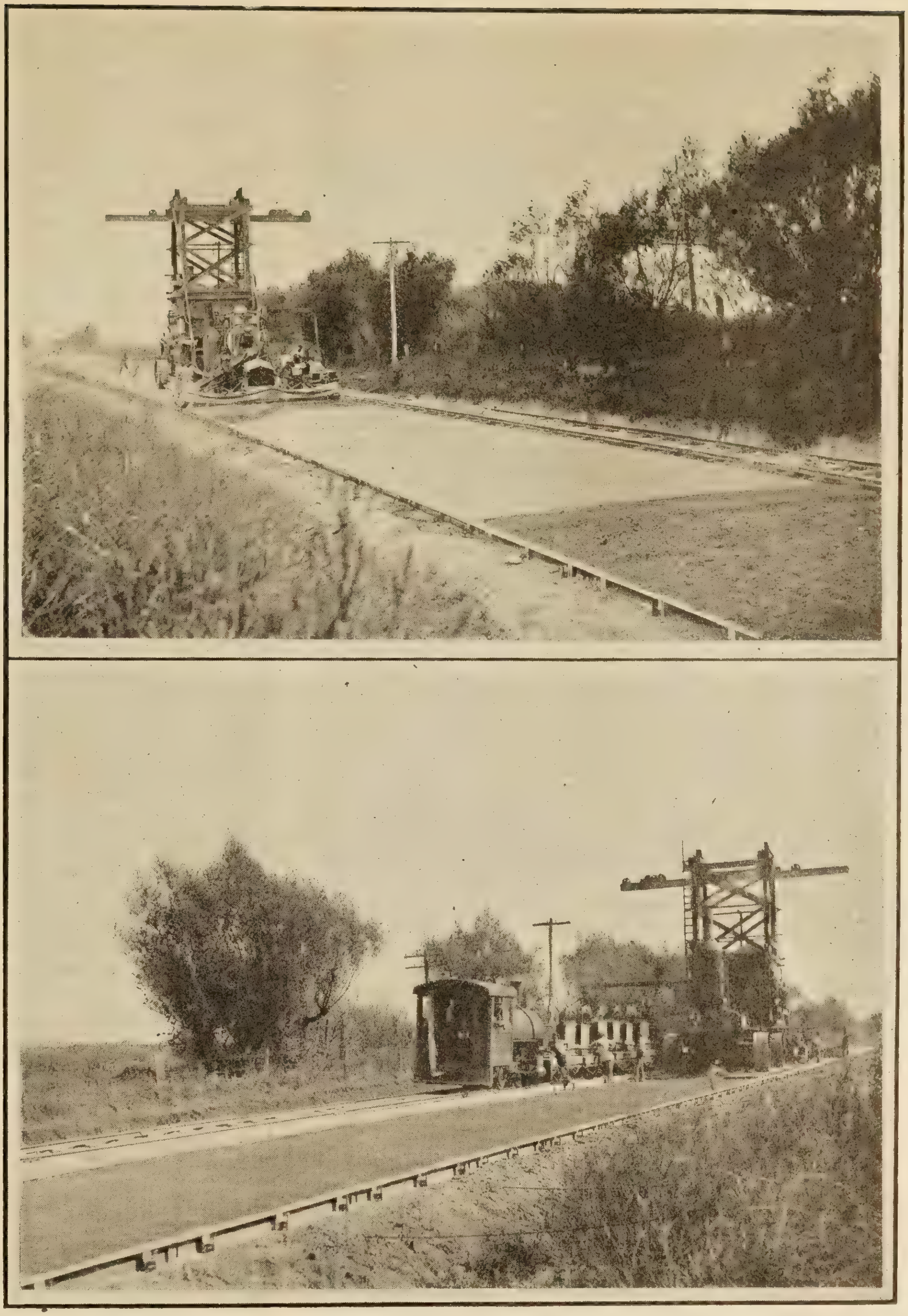
The rest of the equipment at the unloading plant consists of the usual water tanks, pumps, boilers, etc. A one-half cubic yard clamshell bucket is used in unloading the cars and in moving material from the stock piles to the bins. A cement storage shed of 3,000-sack capacity is also provided. There are no unusual features in any of these items.

To operate this plant one derrick operator, a fireman, and two laborers are employed. It has been found in practice that from 15 to 20 cubic yards of aggregate can be handled per hour. The maximum performance with this crew has been 200 cubic yards in one 10-hour shift. After making full allowance for all costs, such as the erection of the plant, depreciation, and interest charges, the cost of unloading material from the cars in which it is received and placing it aboard the cars on which it is hauled to the mixing plants is about 50 cents per cubic yard. This cost may be subdivided as follows: Labor 27½ cents, coal, oil and waste 7½ cents, interest and depreciation 15 cents, total 50 cents.

THE INDUSTRIAL RAILROAD TRACK.

All materials are moved from the unloading plant to the mixing plant on a standard 36-inch industrial railroad track. The equipment used consists of three 16-ton engines and 24 flat cars and a full set of steel-bottom dump buckets of 35 cubic feet capacity. The rails used in the industrial track are standard 40-pound rails. They are laid on 4-inch by 6-inch by 5-foot hewed pine ties, 14 ties to each rail length. The laying of the track cost about $350 per mile and taking it up for relaying costs about $175 per mile. These comparatively low costs are, of course, made possible by the fact that the track is laid on the shoulder of the highway, which made it possible to place the track with almost no expense for preparing the subgrade. On the other hand, the liberal use of ties, though somewhat expensive, is fully justified by the fact that it facilitates track alignment and reduces the labor of maintenance. During the progress of the work no time has been lost on account of derailments or other difficulties chargeable to the condition of the track.

The standard train consists of seven cars, one of which may be loaded with cement, two with sand, and four with broken stone. This loading has not always been used, the cement having very generally been sent out a trainload at a time, a method of handling it which seems better suited to this particular job. The process of loading trains is standard for this class of equipment. The empty train is backed to the bins and so placed that six of the material buckets, of which each flat car carries two, are in position under the chutes. After these are loaded the train is shifted to a position which permits of the filling of the remaining buckets. The operation of loading a train takes about five minutes.



MINNESOTA FEDERAL-AID PROJECT NO. 31. TOP—CONCRETE MIXER AND STORAGE BINS, FINISHED PAVEMENT AND TAMPING MACHINE. BOTTOM—STORAGE BINS FOR SAND AND STONE AT MIXER, HOISTER DEVICE FOR UNLOADING CARS TO BINS.

On this project it has been found that with two trains operating over a 6,000-foot haul and laying about 300 linear feet of pavement per shift, the trains are idle about 30 per cent of the time. No time has been lost on account of accidents. From this it is calculated that the material needed at the mixing plant can be supplied by two trains as long as the haul remains less than 2 miles, and that three trains will serve the mixing plant at a distance of 3 miles from the bins. On this particular project, as the grades are light, it is probable that three trains can serve the mixer at a considerably greater distance than 3 miles, for trains of the length here noted as standard can be operated on a 6 per cent grade, so of course, where the grades are as nearly level as they are on this project longer trains would be more feasible than additional trains.

OPERATING THE TRAINS.

Under the operating conditions prevailing on this project it has been found that a locomotive consumes about 1,000 pounds of coal per locomotive shift. The crew consists of one locomotive driver and one brakeman per train, and operated in this way, including the cost of laying, maintaining, and moving the track, as well as all depreciation and interest charges on the equipment used, the cost of handling aggregate is about $2 per cubic yard for a 3-mile haul. In the nature of the case the cost can not be reduced greatly for shorter hauls. It would be somewhat increased by longer hauls.

One of the most important features in the operation of equipment of this kind is the placing of the switches. On this project the switches are placed about a mile apart. It is doubtful, however, as to whether this practice will prove to have been as economical as the practice of adjusting the position of the switches to the time required in loading, unloading, and moving the trains. There is, of course, a distinct difference in the manner in which the trains must be operated on different projects, depending principally on the methods used in loading and unloading the number of cars on hand. If cars enough are available so that an empty train can be left for loading and a train of loaded cars taken out at once, the standing time of the engine at the loading bins is reduced and the proper position of the switches correspondingly modified. In any event, an arbitrary location of the switches is usually undesirable, as it tends to increase the standing time of the trains. A more advisable practice would seem to be to calculate the average time of loading, the average time of unloading, and the running time between loading and unloading points, and regulate the position of the switches accordingly.

TRAVELING STORAGE BIN.

The unusual feature of the equipment used on this project is the traveling storage bin designed to serve the mixer. This bin, or bins, for there are two compartments, are built on a large platform carried on four wide-tired wheels, the combined storage capacity of the two bins being 12 cubic yards.

Over the bins there is erected a traveling crane, consisting of a light hoist which runs on a heavy beam supported over the bins, and which is operated by means of an engine and drums carried by the platform. By means of this device the buckets delivered over the industrial railroad tracks are lifted off the cars, placed over the bins, and dumped, the coarse aggregate into one bin, the fine aggregate into the other. On the platform also are the propelling engine and an upright boiler which furnishes steam for the hoisting and propelling engines and for the mixer engine as well.

The bins discharge directly into the loading skip of the concrete mixer. This loading skip is divided into two compartments, one of 6 cubic feet capacity for sand and the other of 12 cubic feet capacity for stone. These compartments are filled by opening ordinary gates in the bins of the traveling platform. The process is therefore extremely simple and the accuracy of the measurement easily gauged. Cement is placed in the skip by hand from a flat car alongside the mixer.

The concrete plant and its operation are standard, and therefore require no comment. All finishing work is done by a mechanical tamper of standard make and design.

COMPARISON OF LABOR RESULTS POSSIBLE.

This plant is interesting not only in its design, but because the manner in which the materials are handled makes possible an exact comparison between the force employed and that which would have been required had old hand-labor methods been employed. Thus on this job the moving platform replaces wheelbarrow loading; transportation of materials by industrial railway is substituted for the more common transportation by team or truck; unloading materials with a crane replaces the older methods of hand labor; and the force at the mixer is directly comparable with the operating force ordinarily employed in handling materials, manipulating the mixed material, setting forms, etc. On this project the crew at the mixer consists of a hoisting engineer, a fireman, 1 mixer engineer, 1 man on the sand and stone chutes, 2 men handling cement, 2 men spreading concrete, 1 man operating the finishing machine, 4 men employed in removing forms, watering pavements, etc., and 2 men setting forms, making a total crew of 1 foreman and 15 men. This crew, as suggested above, is directly comparable with the standard crew employed where no loading devices are used, and the saving in labor offers a direct measure of the value of this method of handling materials.

The equipment has worked satisfactorily when the subgrade is dry, but in this particular case the heavy load concentrated on the four wheels of the traveling bins has been too heavy for easy manipulation after the subgrade has been saturated by a number of days of continuous rain. To avoid this difficulty, it is suggested that somewhat smaller hoppers might be more advisable, and that it would also be advisable to distribute the load on a larger number of wheels, thus reducing the unit load so that the subgrade would not be rutted as the hoppers are moved forward.

The advantages of this method of operation, aside from the general advantages which have been noted, are that it eliminates storage of materials on the subgrade and thus preserves the materials in better condition for use, and that it prevents, to a degree, the rutting of the subgrade, which is an inevitable concomitant of delivery of materials by team or motor truck. As tending to offset these advantages there is, of course, the cost of constructing the traveling platform and of moving it over the highway, which will, it is suggested, be largely, if not entirely eliminated wherever the grades are steep. It would therefore seem that this method, though interesting, and on this project successful, should be adopted only after a careful consideration of the conditions which will be met on other projects, and that it will be found to be inapplicable wherever adverse grades of more than 2 or 3 per cent are to be encountered.

The project, as noted above, is being constructed by A. Guthrie & Co., of St. Paul, with A. Lizee, superintendent in actual charge of the work. H. L. Wardell is the engineer in charge of the work for Kandiyohi County.

CONCRETE PRESSURE AGAINST FORMS

By Earl B. SMITH, Senior Assistant Testing Engineer, U. S. Bureau of Public Roads.

THE cost of forms for concrete work constitutes in many cases a large percentage of the total cost of the finished structure, and this cost can only be kept within reasonable limits by rationally studied design methods. In so many cases the form is not designed, but is merely laid out by guess and constructed by the carpenter, with the result that an unwarranted amount of lumber has been used to prevent failure or spreading. The dimensions and the spacing of the supports and braces should receive careful attention to secure sufficient stiffness and ample strength. The sheathing and bracing should be so proportioned as to secure ample stiffness against springing and misalignment. Mere strength without ample stiffness and rigidity is not sufficient for good work.

The proper design of forms can not be effected without knowing the lateral and vertical pressures of plastic concrete against the forms. To secure this information the United States Bureau of Public Roads has made a few tests which seem to accord in general with the results obtained by others,[1] but which go further in indicating the values of some of

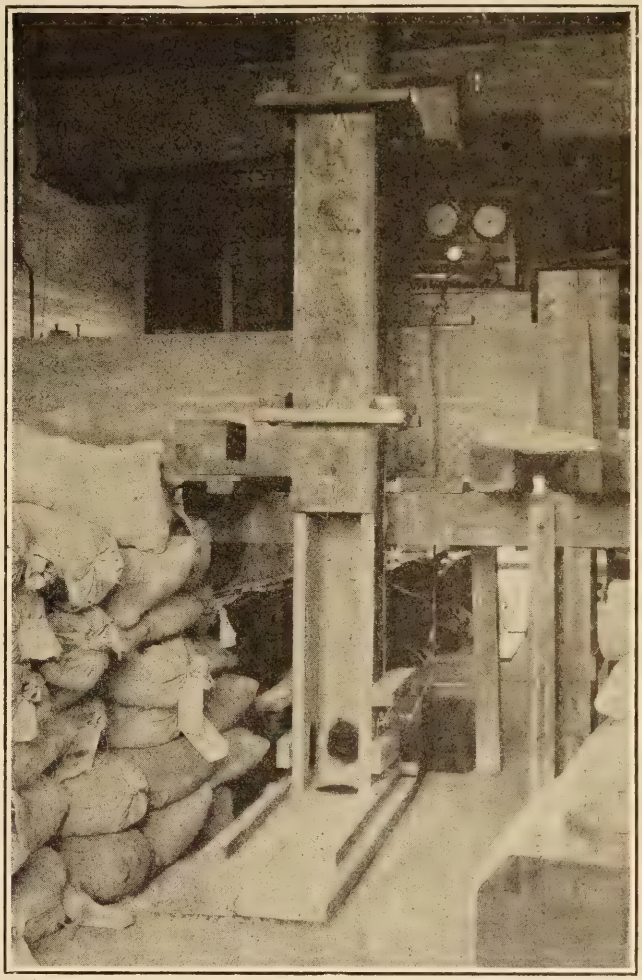


FIG. 2.—SHOWING FORM AND LOCATION OF PRESSURE CELLS.

the factors influencing the results. At the present time sufficient data have been obtained to make any final statement as to the law of pressure of concrete and the effect of each factor, but rather than hold this data longer with the expectation of making it more complete at some future date, it is now offered with the hope that it may serve to make a little more definite the usual practice in the design of concrete forms; also that it may suggest a needed field of investigation for other experimenters.

TESTS TO DETERMINE PRESSURE.

The series of tests presented in this paper was carried out by Mr. W. E. Rosengarten in the laboratory of the research section of the Bureau of Public Roads located at the Arlington Experimental Farm, near Washington. The field tests were made during the construction of the walls and columns of a reinforced concrete building.

The apparatus used to measure the concrete pressures were cells and gauges similar to those described

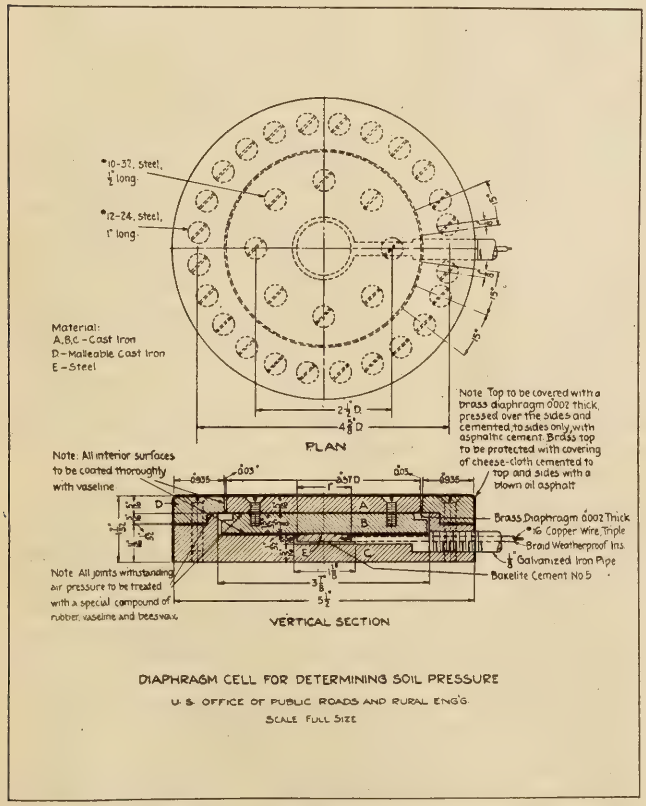


FIG. 1.—DIAGRAM CELL FOR DETERMINING SOIL PRESS.

[1] Design of Concrete Forms, R. A. Sherwin, Eng. Record, Feb. 26, 1916, p. 278; Pressure of Concrete on Forms, F. R. Shunk, Eng. News, Sept. 9, 1909, p. 288; Pressure of Wet Concrete on the Sides of Column Forms, A. B. McDaniel and N. B. Grover, Eng. News, May 18, 1916, p. 933.

in the Proceedings of the American Society for Testing Materials, 1917, page 641, and used for the past few years by this laboratory in measuring earth pressures behind retaining walls and under fills. Details of the instrument are shown in the accompanying figure 1. It consists essentially of an air-tight metal cell having a circular weighing face 10 square inches in area. The concrete pressures against the face of the cell are balanced by admitting compressed air to the inside of the cell. When the pressures are balanced an electrical contact is broken which extinguishes a light and indicates that the pressures shown on the gauge connected with the air pipes is equal to the pressure of the concrete. Tests on these cells show them to be accurate considerably beyond that necessary for these tests, and that the movement of the face is less than one ten-thousandth of an inch to break contact, thus making the cell admirably suited for tests of pressures exerted by granular materials, such as soils, mud, and concrete.

SUITABLE APPARATUS IMPORTANT.

Several other experimenters have attempted to obtain such data, but some have been greatly handicapped by not having a suitable apparatus for determining the concrete pressures. Any scheme for determining the pressure values that depends upon a movement of the concrete at the time of making the readings is evidently not reliable. The values desired are the static pressures of concrete against an immovable surface, and not the pressures necessary to stop a moving mass of concrete, nor to start a movement of the mass. And any scheme requiring first the disturbance or movement of the mass before making the pressure readings is also undesirable.

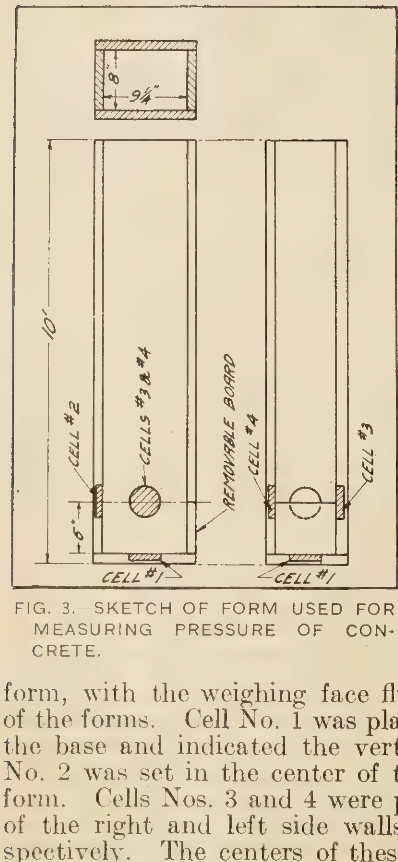


FIG. 3.—SKETCH OF FORM USED FOR MEASURING PRESSURE OF CONCRETE.

The arrangement of the pressure cells and the concrete form used during the laboratory tests is shown in the accompanying figures 2 and 3.

The form was built of 2-inch planks giving an inside horizontal cross section of 7.8 inches by 9.4 inches, and a height of 10 feet.

Four cells were placed in the form, with the weighing face flush with the inside of the forms. Cell No. 1 was placed in the center of the base and indicated the vertical pressure. Cell No. 2 was set in the center of the rear wall of the form. Cells Nos. 3 and 4 were placed in the center of the right and left side walls of the forms, respectively. The centers of these cells for obtaining the lateral pressure were all 6 inches above the base. Air-control pipes leading from all cells were arranged in order, with connections and nipples conveniently located for taking the readings on the four cells very quickly.

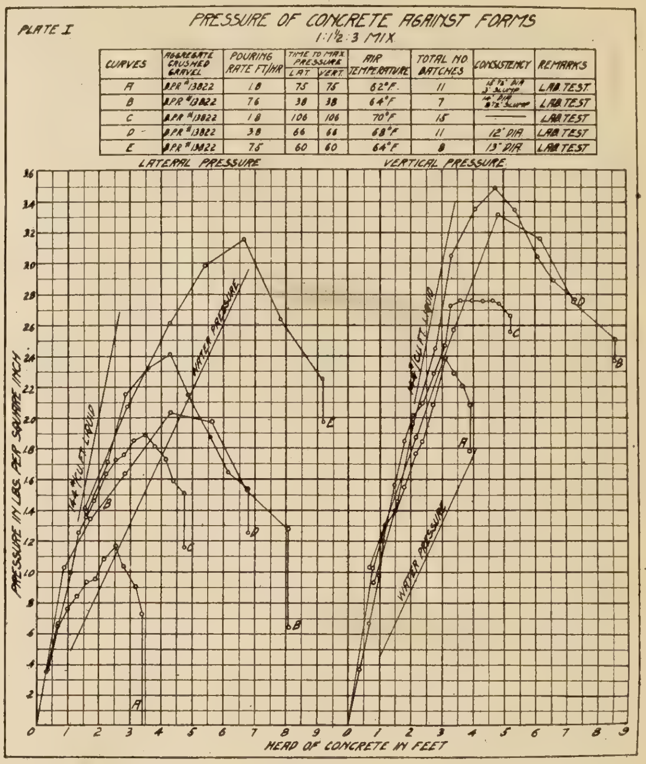


FIG. 4.—LABORATORY TESTS.

MAKING THE TESTS.

The materials for the concrete used in the tests were carefully weighed and mixed by hand. Immediately upon completing the mixing the concrete was shoveled into buckets and dumped into the top of the forms. The mixing floor and the wood forms were well wetted before the test was begun. The concrete was tamped on top by the use of a long stick having a 2-inch by 6-inch foot on the lower end. The forms were also vibrated by striking the outside with a heavy hammer. The height to which the concrete stood in the column form was then measured, and the pressures on the bottom and three side cells were immediately read and recorded. The batches were varied in size so that when a new batch was added each 10 minutes the head of concrete in the form would increase at the rate desired. Readings were taken on the pressure cells immediately after placing the concrete and again about five minutes later, or shortly before placing the next batch of concrete.

The air and the mixing water temperatures were recorded each day tests were run. Slump tests of the concrete were made to determine the consistency used in each test; and are recorded as inches slump, or where very wet as inches diameter of the mass. Fresh batches of concrete were added every 10 minutes until after the pressures on the cells had passed a maximum and indicated a decided decrease in pressure. The tabulated data and results of these laboratory tests are shown in Table I, and also, in graphical form, in figures 4 to 7, inclusive.

Several field tests were run, in addition to the laboratory tests described above, during the construction of a reinforced concrete building at the

Arlington Farm. The pressure cells were inserted in the wall and column forms as shown in the accompanying figures 8, 9, and 10, and pressure readings taken at the time the concrete was being poured. The concrete was machine mixed, raised in an elevator, and directed into the forms through a system of chutes. It was then spaded or tamped with a stick with a small blade on the end. The concrete was a 1 : 2 : 4 mix, river gravel being used for the coarse aggregate, and the consistency rather sloppy, flowing readily around the steel reinforcing. The results from these field tests conform favorably with those

	Per cent.
Passing 50-mesh, retained on 80-mesh sieve	15.0
Passing 80-mesh, retained on 100-mesh sieve	3.0
Passing 100-mesh, retained on 200-mesh sieve	6.4
Passing 200-mesh	7.8

The fine gravel (Laboratory No. 14192) consisted of subangular fragments of quartz with some chert and sandstone. The grading of this gravel is as follows:

	Per cent.
Passing 1½-inch, retained on 1-inch screen	2.8
Passing 1-inch, retained on ¾-inch screen	6.6
Passing ¾-inch, retained on ½-inch screen	29.2
Passing ½-inch, retained on ¼-inch screen	47.9

Table I.—*Tests of pressures of concrete against forms.*

Test No. data.	Mix.	Aggregate.	Consistency.	Temperature.		Rate of pouring per hour.	Maximum pressure.		Head at maximum pressure.		Time at maximum pressure.	
				Air.	Water.		Vertical.	Lateral.	Vertical.	Lateral.	Vertical.	Lateral.
1918.				°C.	°C.	Feet.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
June 17	1:2¼:3¾			23		1.50	1.15	0.68	2.50	2.00	90	90
June 18	do			26		2.70	1.45	.90	3.70	2.50	78	61
June 19	do					3.30	1.65	.73	3.30	2.50	49	44
Nov. 23	1:3:6	Cement, 13820	Quaky	19	21	1.63	13.50	.52	2.77	1.70	79	61
Nov. 26	do		Quaky (forms)	15	18	3.32	1.05	.43	4.15	2.53	70	50
Nov. 30	do		Quaky (dry)	19	20	6.72	1.07	.21	6.59	3.80	57	37
Dec. 4	do		Quaky	17	19	14.26	1.33	.41	10.00	5.12	48	29
Dec. 9	do		do	17	21	[1] 9.00	.86	.35	9.28	8.78	10	10
Jan. 17	do	Gravel, 13822	do	21	19	1.73	.98	.90	1.71	2.08	36	66
Jan. 13	do	New lot cement	Quaky (first test, forms wet)	18	20	3.46	2.02	.85	3.72	3.22	57	57
Jan. 14	do	do	Quaky	16	19	7.08	2.53	1.40	5.50	5.00	49	49
Jan. 31	1:2:4	Gravel, 13822	Quaky (14-inch diameter)	19	18	1.75	2.27	1.50	3.46	2.68	96	86
Mar. 1	do	New cells	Quaky (13-inch diameter)	24	18	3.60	2.35	1.46	4.50	3.40	66	56
Mar. 3	do	do	do	23	23	7.32	1.73	1.21	4.55	4.05	37	37
Jan. 18	1:1½:3	Gravel, 13822	Quaky	21	24	1.86	2.75	1.89	3.99	3.49	106	106
Jan. 20	do	do	Quaky (12-inch diameter)	20	17	3.84	3.49	2.40	4.79	4.29	66	66
Jan. 21	do	do	Quaky (13-inch diameter)	17	17	7.50	4.60	3.16	7.10	6.60	60	60
June 2	1:2:4	Fine gravel, No. 14192	Quaky (10½-inch diameter)	25	18	1.78	1.75	.98	2.09	1.87	46	56
June 3	do	do	Dry (6½-inch slump; 11½-inch diameter, 6½-inch slump).	27	18	7.35	2.41	1.93	3.33	2.83	28	28
Apr. 10	1:1½:3	Gravel, 13822	Sloppy (12½-inch diameter, 3-inch slump).	16	15	1.84	2.37	1.17	3.05	2.55	75	75
Apr. 12	do	This test and all previous crushed gravel, 13822.	Sloppy (14-inch diameter, 8½-inch slump).	17	15	7.66	3.31	2.05	4.80	4.30	38	38
Apr. 18	1:2:4	First test, fine gravel, 14192	Sloppy (17-inch diameter, 9½-inch slump).	18	17	1.86	2.75	1.63	3.38	2.88	85	85
May 9	do	do	Sloppy (16-inch diameter, 9½-inch slump).	19	17	7.44	1.97	1.19	3.33	2.83	27	27

[1] In one batch.

Table II.—*Field test of pressure of concrete against forms.*

Test No. data.	Mix.	Consistency.	Temperature.		Rate of pouring per hour.	Distance cell to opposite side of form.	Maximum pressure, lateral.	Head at maximum pressure, lateral.	Time at maximum pressure.
			Air.	Water.					
			°C.	°C.	Feet.		Lbs.	Feet.	Minutes.
Apr. 15	1:2:4	Sloppy	14	13	12.0	8 inches	1.95	3.25	17
					12.0	do	2.45	3.25	17
Apr. 17	do	do	14	15	20.0	3 inches to reinforcement	[1] 2.45	4.62	23
					20.0	18 inches	[2] 3.90	4.62	23
May 26	do	do	23	23	9.0	18 x 28 inch hole	2.20	3.00	20
May 26	do	do	23	23	12.5	9½ inches	1.85	2.30	11
May 26	do	do	23	23	10.6	do	1.45	2.30	13

[1] Stopped. [2] Pour.

obtained from the laboratory and are shown in Table II, and figure 11.

CONCRETE MATERIAL USED.

As a matter of record, the following information as to the concrete material is given:

Cement, Tidewater brand, passing standard specifications for cement.

The sand (Laboratory No. 13821) was obtained locally and consisted of angular quartz particles with some ferruginous clay and a little mica and magnetite. It showed a loss by washing of 6.6 per cent, with grading as follows:

	Per cent.
All passing ¼-inch, retained on 10-mesh sieve	12.0
Passing 10-mesh, retained on 20-mesh sieve	15.2
Passing 20-mesh, retained on 30-mesh sieve	17.6
Passing 30-mesh, retained on 40-mesh sieve	14.6
Passing 40-mesh, retained on 50-mesh sieve	8.4

	Per cent.
Passing ¼-inch, retained on 10-mesh sieve	11.9
Passing 10-mesh, retained on 20-mesh sieve	.2
Passing 20-mesh, retained on 30-mesh sieve	.1
Passing 30-mesh, retained on 40-mesh sieve	.1
Passing 40-mesh, retained on 50-mesh sieve	....
Passing 50-mesh, retained on 80-mesh sieve	.2
Passing 80-mesh, retained on 100-mesh sieve	.1
Passing 100-mesh, retained on 200-mesh sieve	.1
Passing 200-mesh	.8

Crushed gravel aggregate. (Laboratory No. 13822.) This consisted of large, angular fragments of sandstone with a few rounded quartz pebbles with grading as follows:

	Per cent.
Passing 1½-inch, retained on 1-inch screen	30.3
Passing 1-inch, retained on ¾-inch screen	32.8
Passing ¾-inch, retained on ½-inch screen	18.1
Passing ½-inch, retained on ¼-inch screen	13.8
Passing ¼-inch screen	5.0

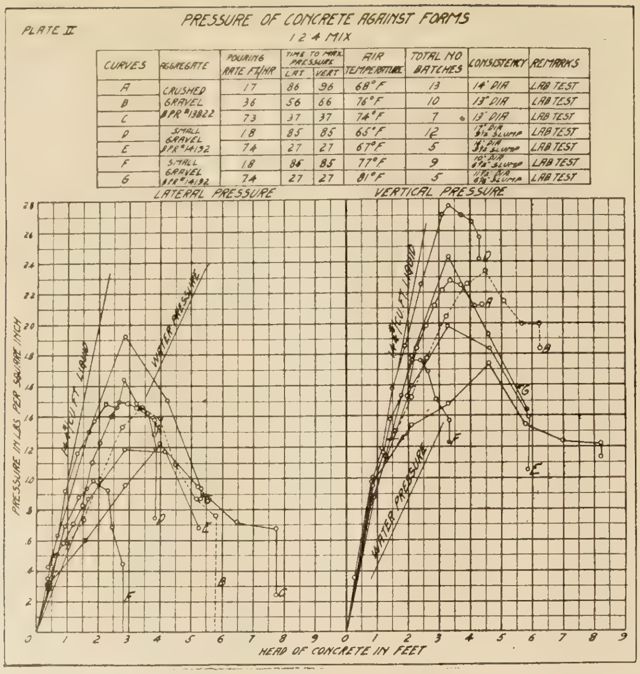


FIG. 5.—LABORATORY TESTS.

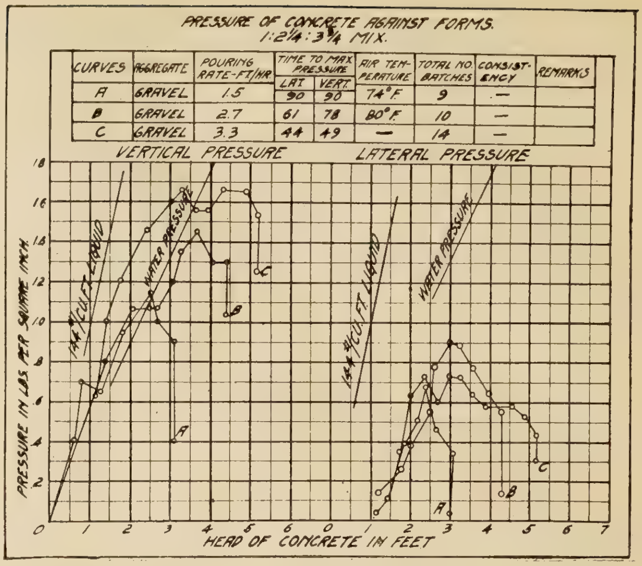


FIG. 6.—LABORATORY TESTS.

FACTORS INFLUENCING PRESSURE.

The results shown by these experiments indicate that the fundamental pressure of concrete against the form is about 1 pound per square inch for the first 1 foot of head. However, this is by no means all that should be said. A study of the results reported by others, and those obtained from this series of tests, shows that the following factors have an influence upon the pressure, namely, (1) rate of filling the forms, (2) cross-sectional area of the forms, (3) consistency of the concrete, (4) amount of cement in the concrete, (5) temperature of the concrete and the time of set and of the cement, and (6) character of the fine and the coarse aggregate.

Sufficient information is not yet available to make final statements as to the law by which each of these factors influences the pressure of the concrete against the form. The results do show that the initial pressure under small heads is equal to the hydrostatic pressure of a liquid having the approximate density or weight of the concrete; that is, approximately 1 pound per square inch, or 144 pounds per square foot, for the first foot head. As pouring is continued this pressure, however, soon falls below the straight line hydrostatic pressure, and the amount of this deviation depends upon one or more of the factors mentioned above.

It is important to notice that the results prove that if filling is continued indefinitely the lateral pressures near the base of the form finally reach a maximum value and then decrease gradually to zero, regardless of the fact that fresh concrete is continually added above. The vertical pressures are in all cases greater than the lateral pressures—they decrease in value after a maximum has been attained, but not to zero. The total weight of the concrete mass in ordinary construction is not supported entirely upon the bottom of the form, but because of the roughness and friction against the sides the planking takes part of the weight or vertical pressure. Of course, for wide and shallow masses of concrete such as floor slabs the vertical pressure is equal to the weight of the concrete.

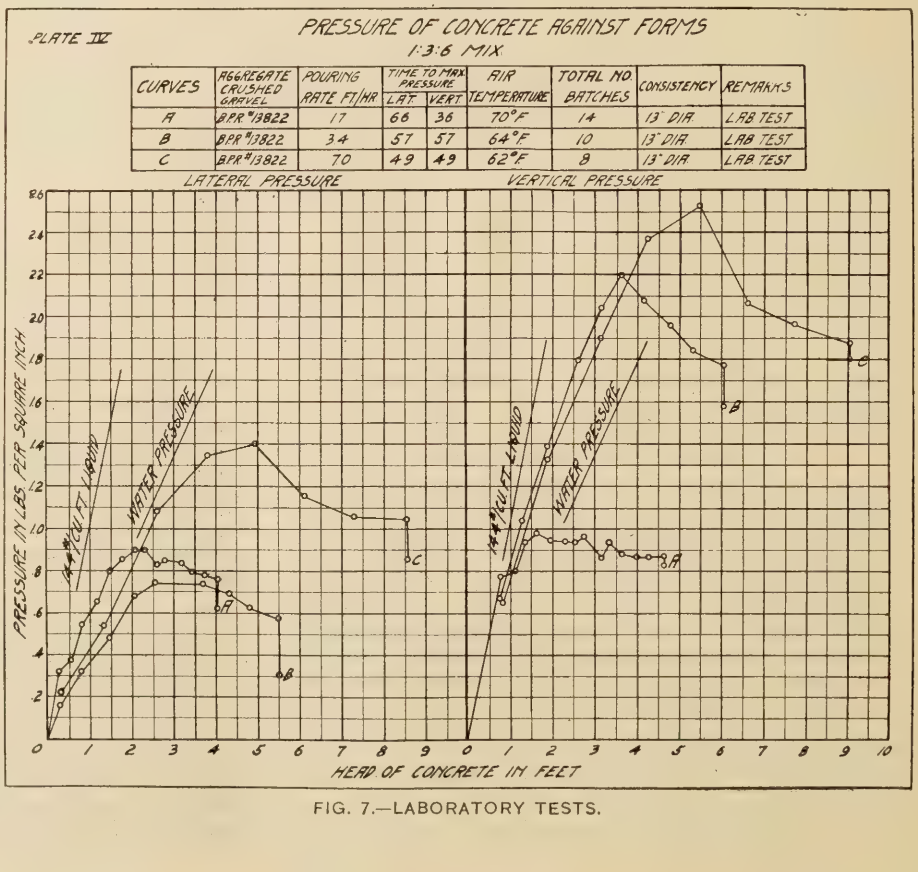


FIG. 7.—LABORATORY TESTS.

EFFECT OF INFLUENCING FACTORS.

A summary of the data at hand seems to lead to the following conclusions regarding the effect of the various influencing factors:

(1) The maximum pressure exerted upon the forms increases as the rate of filling increases. At a slow rate of about 1 foot per hour the pressure is approximately 1 pound per square inch, but as the rate increases beyond this value the pressure increases approximately as the 0.3 power of the rate.

(2) Field tests, which were made in places where the distance between the form walls differed, indicate that the maximum pressures obtained increase slightly with the mass of the concrete when the consistency is wet and sloppy. This conclusion probably does not hold in the case of dry mixes. Reinforcing just inside the form tends to slightly decrease the pressures, but probably this effect should be neglected in determining the final pressures for use in design.

(3) The results show in general that the maximum pressure was increased as the consistency of the concrete was made drier within the limit of workability. This, probably, differs from what might be expected, but the tests show it to be the case. It is probably due to the fact that under the usual conditions of placing dry concrete it requires more tamping, which, because of its dryness, seem to develop a permanent wedging action between the particles. In the case of wet or sloppy concrete this wedging action does not exist, as we have approximately a static fluid pressure. For low heads the dry concrete (when tamped as usual) will give the greater lateral pressure, but for heads of 4 feet or more and within the time when initial set becomes an influencing factor the sloppy mixtures give the greater pressure. The average increase of pressure due to the effect of dry mixtures seems to be 0.3 pounds per square inch for each inch decrease in the standard slump test less than a 5-inch slump.

(4) The richness of the mix also affects the maximum pressures obtained. The richer the mix the greater the maximum pressure. The average increase being 0.12 pound per square inch for each per cent increase in the ratio of the cement to the aggregate beyond 12 per cent.

(5) A decrease in the temperature of the concrete retards the set of the cement, and it is natural to suppose that this is the limiting factor in the maximum pressure obtained, since the pressure increases with the head until the cement takes a sufficient set to begin to support the overlying concrete. Therefore, as the temperature is reduced and the time of setting is increased the height of fill may be increased and thus produce or make possible a higher total pressure. Since the cement begins to set and stiffen in about 30 minutes, the maximum pressure is attained under whatever head of concrete may exist at this time. The value for H, the head of concrete, to be used in the formula given below should not be greater than one-half the rate of fill, except where agitation is vigorous and continuous in a sloppy mix; then this ratio may be taken up to three-fourths.

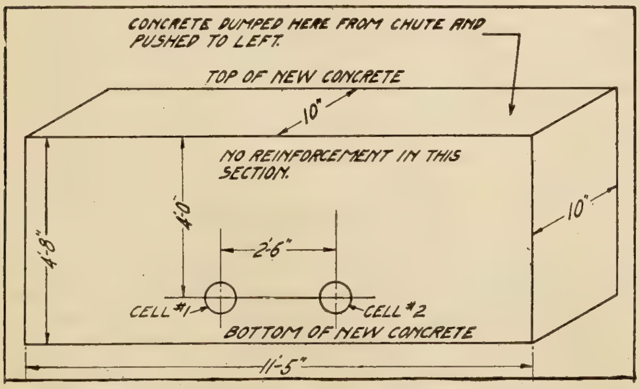


FIG. 8.—SHOWING LOCATION OF PRESSURE CELLS NOS. 1 AND 2, FIELD TESTS, DURING CONSTRUCTION OF REINFORCED CONCRETE BUILDING.

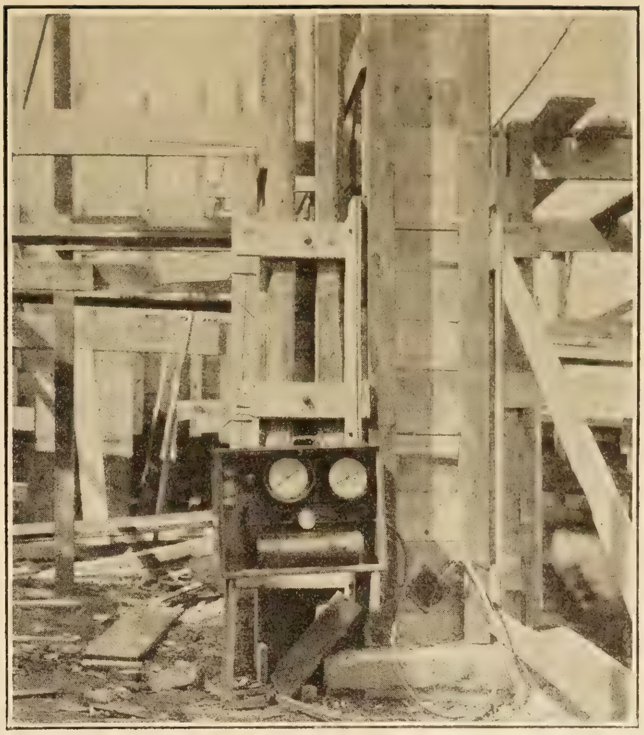


FIGURE 9.—SHOWING POURING OF COLUMN, FIELD TEST.

FORMULAS GIVING PRESSURES.

An empirical formula giving the lateral pressures required for use in the design or the investigation of the strength of concrete forms, and taking into account the above numerical factors, is

$$P = H^{0.2}R^{0.3} + 0.12C - 0.3S,$$

P being the resultant lateral pressure in pounds per square inch; R, the rate of fill in feet per hour; H, the head of concrete fill; C, the per cent by volume of cement to the combined fine and coarse aggregate; and S, the consistency in inches of slump.

The vertical pressure is obtained by adding $0.25H$ to the value of P as found above, except when the inside distance between the vertical sides of the form is greater than one-half the depth of fill; then the value should be taken as equal to the weight of the concrete.

In the practical application of this formula, as with all formulas, there is abundant opportunity for the exercise of common sense and good judgment. The formula may give pressures somewhat higher than exact values. It shows the effect of continuous and vigorous agitation of the concrete mass only as this is introduced through good judgment in selecting the value for the head of concrete, H. For usual conditions H may be taken as not greater than one-half of R. For ordinary cement in cold weather, or when continuously and well agitated, H may be three-fourths of R, when the filling is continuous beyond 1 hour. A second pouring on top of concrete that has been in place for 45 minutes or more does not add to the pressures already existing at the bottom of the fill.

The values for C may be taken as the next higher whole number in the per cent of cement by volume, as the required accuracy does not justify fractional per cents. Values for S may also be taken only as whole numbers, since the slump test is not accurate closer than 1 inch.

The value of P obtained by the formula is the lateral pressure against the form at the lowest point of the fill. Since the pressures are not uniform from top to bottom, but vary approximately as the ordinates of a parabola, the center of pressure or point of resultant pressure may be taken at 0.6 of the height of fill, H, from the top.

USE OF THE FORMULA.

The following examples may serve to show the use of the above formula:

Example I.—For reinforced mass concrete. Mix to be 1:3:5; consistency, rather sloppy, or 9-inch slump; the rate of fill, R, to be 8 feet per hour. The total height of concrete filled within 1 hour, 7 feet. Since this concrete is placed by means of a chute in a large form, and men are continually walking around in it, the value to be chosen for H is 6, or three-fourths of R. Then, substituting in the formula—

$P = 6^{0.2}8^{0.3} + (0.12 \times 13) - (0.3 \times 9)$.
$P = 1.53$ pounds per square inch.
The vertical pressure = 7 pounds per square inch.

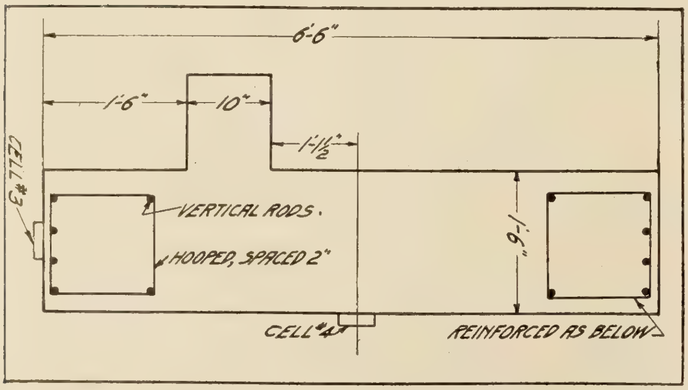


FIG. 10.—SHOWING LOCATION OF PRESSURE CELLS NOS. 3 AND 4 DURING FIELD TESTS.

Example II.—For reinforced concrete column. Mix to be 1:2:4; consistency, 8-inch slump; rate of fill to be 24 feet per hour. Total height of column and final fill, 11 feet, made in one pouring; since this is done in less than 30 minutes the value for H is 11. Substituting in the formula—

$P = 11^{0.2}24^{0.3} + (0.12 \times 17) - (0.3 \times 8)$.
$P = 3.83$ pounds per square inch.
Vertical pressure $= P + 0.25H = 6.58$ pounds per square inch.

PLATE V

CURVES	AGGREGATE	POURING RATE FT/HR	TIME TO MAX PRESSURE LAT.	VERT.	AIR TEMPERATURE	TOTAL NO. BATCHES	CONSISTENCY	REMARKS
H	GRAVEL	12.0	17	–	58°F	–	SLOPPY	FIELD TEST
I	GRAVEL	12.0	17	–	58°F	–	SLOPPY	FIELD TEST
J	GRAVEL	20.0	23	–	74°F	–	SLOPPY	FIELD TEST
K	GRAVEL	20.0	23	–	74°F	–	SLOPPY	FIELD TEST
L	GRAVEL	18.0	11	–	74°F	–	SLOPPY	FIELD TEST
M	GRAVEL	8.0	13	–	74°F	–	SLOPPY	FIELD TEST

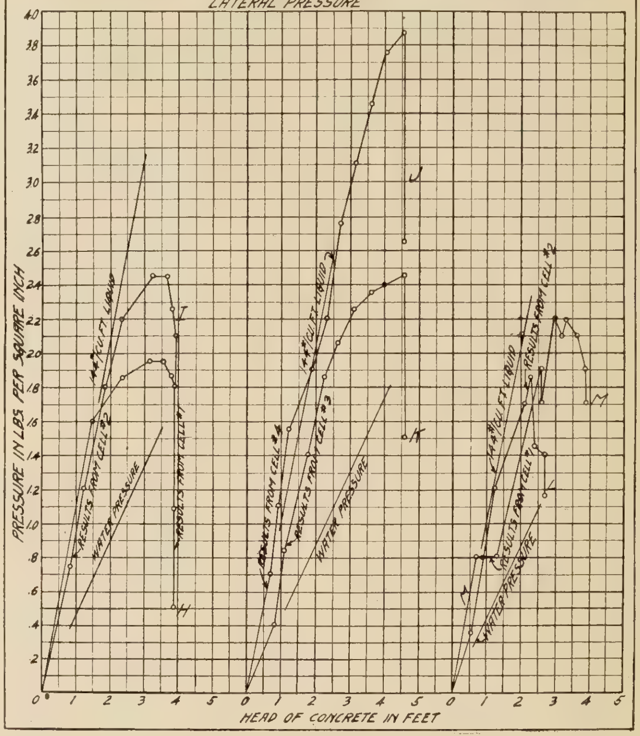


FIG. 11.—FIELD TESTS.

Example III.—For dry mix, mass concrete. Mix to be 1 : 3 : 6; consistency, 3-inch slump; rate of fill 6 feet per hour; distance between sides of form, 3 feet; total height of fill within 30 minutes, 4 feet. Then,

$P = 4^{0.2}6^{0.3} + (0.12 \times 11) - (0.3 \times 3)$.
$P = 2.68$ pounds per square inch.
Vertical pressure = 4 pounds per square inch.

ALABAMA'S BOND ISSUE.

The people of Alabama, at the special election in February, adopted by a large majority the constitutional amendment permitting the issue of $25,000,000 bonds. In only one county in the State was there a majority against the amendment. On the same day Jefferson County, that State, voted in favor of an issue of $5,000,000 of bonds for highway construction.

OREGON BOND ACT.

At its special session in January of this year the Oregon Legislature passed a road-bonding act providing for $10,000,000 in bonds for State highways, a constitutional amendment permitting the issue to be submitted to the people at a special election on May 21. The proposed amendment increases the limit of the State's bonded indebtedness from 2 to 4 per cent of the assessed valuation.

PROPER CONSISTENCY OF BITUMINOUS MATERIALS IN HIGHWAY ENGINEERING

By PRÉVOST HUBBARD, Chemical Engineer.[1]

AS applied to bituminous materials for use in highway engineering the term consistency is most commonly used to designate "degree of firmness," which is one of Webster's definitions of the word consistence. Both terms are very broad as applied to the physical properties actually determined by the common tests for consistency, so broad in fact that results obtained by one method can not be accurately translated into the results obtained by any other method.

Before considering these methods and their significance, it should first of all be realized that the true bitumen of bituminous highway materials is in reality a fluid, although it is called semisolid or solid when its degree of firmness passes certain arbitrary points. The one certain property of any fluid is that under certain conditions it will flow. Degree of firmness is therefore indicated by degree of fluidity, most readily determined by "resistance to flow," which is the accepted definition of the term viscosity. Most of the direct tests for determining the consistency of bituminous materials are therefore viscosity tests, although they may be made in a variety of ways.

The resistance to flow of any bituminous material is substantially influenced by the temperature of the material, a general rule being that the higher its temperature the lower becomes its resistance to flow. While this is true, the relative decrease in resistance to flow with increase in temperature is not necessarily the same for all bitumens and in fact may be widely different for different classes or types of bitumen which may show identically the same resistance to flow at a given temperature. Put in other words, some bituminous materials are more susceptible to temperature changes than others, and in general their consistency at a given temperature is no certain indication of their consistency at another temperature.

Bituminous materials may contain other material than bitumen and almost invariably, as used in highway treatment and construction, are purposely mixed or become mixed with mineral matter. The amount, character, and fineness or grading of the mineral matter may exert a tremendous influence upon the consistency of the mixture, and if the mineral particles are in sufficient quantity to be in contact their state of compaction is also an important factor. In the ultimate analysis, degree of firmness of such mixtures as they exist in the highway is the primary consideration, and selection or control of the consistency of the bitumen present is only one of a number of contributing factors in obtaining the desired degree of firmness, more often ex-expressed as resistance to displacement of the mixture If this reasoning is correct, it is manifestly irrational to limit the consistency of a bituminous material with any greater degree of refinement than the control of other equally important factors bearing upon the resistance to displacement of the bituminous mixture.

METHODS OF DETERMINING CONSISTENCY.

Coming now to the methods determining the consistency of bituminous materials, we have first to consider the grade and type of material. So far as method of use is concerned, it is convenient to establish three general classes: (1) Materials for cold surface treatment, (2) materials for hot-surface treatment, and (3) materials for construction. The types are (1) petroleum and asphalt products and (2) tar products.

The consistency of both types for cold-surface treatment is most frequently determined and specified by a viscosity test made with the Engler viscosimeter, which is used to measure the time required for a given quantity of the material to flow through a standard tubular opening under a standard initial head. For such use the first consideration is to have the material sufficiently fluid under ordinary atmospheric conditions to insure its uniform distribution over the road surface at the proper rate per square yard according to the method of distributing. For this purpose a maximum viscosity limit should be used. For materials to be used solely as dust paliatives no minimum viscosity limit is necessary as resistance to displacement of its admixture with dust particles does not have to be taken into account. Petroleum products are used almost exclusively as dust palliatives and should be sufficiently fluid to apply by means of a gravity distributor if necessary. A maximum specific viscosity of 10 at 25° C. will insure the desired degree of fluidity, and this is the only consistency requirement which is necessary.

When a bituminous material is to be applied cold for the purpose of building up a thin mat or carpet with a cover of mineral matter the ultimate stability

[1] Paper read before meeting of State highway testing engineers and chemists held at Washington, Feb. 23–27, 1920.

of such a mat becomes an important quality. With the average mineral cover of broken stone or sand no cold application material possesses the desired degree of firmness, but it should approach this characteristic as closely as possible and should develop it to the fullest extent soon after application. As in the case of dust palliatives, a maximum viscosity limit is necessary to insure that it is sufficiently fluid to be applied at normal atmospheric temperatures. When application is made by means of a pressure distributor it has been found that a maximum specific viscosity of 120 at 25° C. is about the safe limit to use in the case of oil products, and a range of from 80 to 120 specific viscosity is considered reasonable. Cut-back asphalts, which harden rapidly through loss of a relatively small amount of volatile flux, may properly be held to a very much lower range, and for this class of material the American Society for Municipal Improvements has adopted limits of from 25 to 35 specific viscosity at 25° C. The reason for this is that allowance must be made for possible hardening during handling before application is made. Thus, if a maximum of 120 specific viscosity was allowed for cut-back asphalts, unavoidable loss of volatile constituents before application might increase the consistency of the product to such an extent that it could not be successfully applied cold.

VISCOSITY OF TAR PRODUCTS.

Tar products for cold surface treatment are seldom used for the purpose of dust laying only. For mat or carpet construction it has become customary to specify their specific viscosity at 40° C. rather than at 25° C. This is purely a matter of convenience, however, from the standpoint of testing, as the same factors govern their viscosity limits as mentioned for oil products. Tars are more susceptible to temperature changes than are the oils, and their susceptibility factors vary considerably. It has been found, however, that for tars containing not more than the usual maximum limit of 10 per cent free carbon a specific viscosity at 40° C. of over 35 is apt to cause difficulty in distribution even in warm summer weather. This limit is, therefore, the maximum that should be allowed. A safe minimum for cool weather has been found to be 10 specific viscosity at 40° C. Within these limits the American Society for Municipal Improvements recommends that a range of 5 be allowed for any one job. Limits of from 10 to 25 for cool climates and from 20 to 35 for warm climates would, however, appear to be reasonably satisfactory for ordinary use.

Tars harden with relative rapidity upon exposure under atmospheric conditions, sometimes to the extent of becoming too brittle to give long service. On the other hand, petroleum products may not harden with sufficient rapidity to produce the desired resistance to displacement in the bituminous mat. In order to insure the necessary hardening properties of the latter, a consistency requirement for the residue obtained from the volatilization test is sometimes included in specifications. Thus for oil products the United States Bureau of Public Roads specifies a minimum float test of 90 seconds at 50° C. on the residue from the volatilization test, while for cut-back asphalts the American Society for Municipal Improvements specifies a penetration at 25° C. of from 50 to 85 for such residue. The maximum limit in the latter case is to prevent the use of too hard an asphalt in the manufacture of the cut back. No maximum limit is required for petroleum products owing to the presence of nonvolatile oils, which prevent undue hardening after application.

IN HOT-SURFACE TREATMENT.

For hot-surface treatment much more viscous materials may be used than for cold-surface treatment, but they should be sufficiently fluid at the temperature of application, 95° to 130° C., to be uniformly distributed at the proper rate, and before cooling after application should be fluid enough to saturate the surface of the road and firmly adhere to it. For oil products this will require a maximum specific viscosity at 100° C. of not more than 60 if the surface treatment of gravel as well as macadam roads is to be included. All tar products of suitable normal consistency are sufficiently fluid, at the temperatures of application previously mentioned, to apply satisfactorily, so that a consistency test at such range of temperatures is unnecessary. At normal temperature both oil and tar products for hot-surface treatment should be too viscous to be conveniently tested with the Engler viscosimeter, and their consistency is probably best controlled by means of the float test at 32° C. In view of the maximum viscosity limit at 100° C. only a minimum float test at 32° C. need be specified for petroleum products without danger of securing too hard a material. For this purpose the United States Bureau of Public Roads has set a minimum float test at 32° C. of 60 seconds. The consistency of tar products for the same purpose, however, is limited by a minimum float test of 60 seconds and a maximum float test of 150 seconds at 32° C. in order to prevent the use of an undesirably soft or undesirably hard product. For reasons mentioned in discussing the cold application materials, specifications for petroleum products for hot application may include a minimum consistency requirement for the residue from the volatilization test. This limit may reasonably be somewhat higher, as illustrated by the United States Bureau of Public Roads specification in which the residue is required to show a float test at 50° C. of not less than 110 seconds, thus insuring a material which will harden after application.

USE OF BITUMINOUS MATERIALS.

Bituminous materials for highway construction are used according to the penetration method or the mixing method to coat the particles of mineral aggregate of the pavement throughout a depth of seldom less than 2 inches. In the former case the material is applied to the pavement proper and while still hot must flow between the mineral fragments to the required depth and coat the exposed surfaces. Upon coming in contact with the broken stone a portion of it almost immediately congeals on the cold surfaces. The coatings of bitumen are therefore relatively thick as compared with coatings produced by the mixing method of construction. Stability of such pavements commonly known as bituminous macadam depends to a very great extent upon the interlocking of the large mineral fragments. Under traffic there is, however, more or less internal movement, so that eventually the bituminous material finds its way between the corners, edges, and surfaces in actual contact, although the coatings at such places are necessarily squeezed thinner than at other places. For this reason the bituminous cement should be sufficiently hard under conditions of service to prevent it from acting as a lubricant. On the other hand, it should be sufficiently soft to heal fractures or displacements due to internal movement. These factors are of such importanceas to warrant limits of consistency based upon general climatic conditions to which the pavement will be subjected.

Because of the fact that tars harden materially after use they are required to be of softer original consistency than are the asphalt cements used for a similar purpose. They should be so soft in fact that they can not be tested by means of the penetration test made in the usual manner. Their consistency is usually specified by a float test at 50° C., which is approximately the maximum temperature the pavement is likely to attain. For general low-temperature conditions a float test at 50° C. of from 120 to 150 seconds is commonly specified and for high temperature conditions these limits are raised to from 150 to 180 seconds.

CONSISTENCY OF ASPHALT CEMENT.

Owing to their rapid hardening, especially when existing in thin films, and to their high susceptibility to temperature changes, tars are now seldom used in the mixed types of construction. Asphalt cements are almost exclusively used for such types, and as selection of their consistency limits for asphalt macadam, the various classes of asphaltic concrete, sheet asphalt, and other mixed types, involves many considerations common to all, they may best be considered collectively.

What may be termed the normal consistency of asphalt cements is determined and specified by means of the penetration test, the factors of temperature, loading, and time being 25° C., 100 grams, 5 seconds. In this test, degree of firmness is determined by recording the distance that a standard needle penetrates a sample of the material under the above-mentioned conditions of temperature, loading, and time.

While certain rather wide limits of normal penetration have been found to describe suitable consistencies of asphalt cements for the various types of construction, no absolute standards of limits have been generally adopted for each type under accurately defined temperature and traffic conditions. It is recognized, however, that for any given type under otherwise similar conditions the warmer the climate the lower should be the normal penetration of the asphalt cement. In like manner, considering traffic as the only variable, it is recognized that the heavier the traffic the lower should be the penetration of the asphalt. As far as the types themselves are concerned, for all sheet or continuous forms of construction in which only a relatively small percentage of mineral particles pass the 200-mesh screen, it is also recognized that the finer the aggregate the lower should be the penetration of the asphalt cement. In connection with this statement, fineness of aggregate is meant to imply a weighted average diameter of fragment.

CLIMATE AND TRAFFIC CONDITIONS.

At the present time climatic and traffic conditions with reference to highways are dealt with under very broad general terms, which unfortunately do not always have the same significance in the minds of different individuals, owing to the fact that the terms are merely relative. Thus the terms low, moderate, and high are used to describe general temperature conditions. To the average mind it would seem that low temperature conditions might imply climates in which the winters are long and severe while the summers are relatively short and temperate. Moderate temperature conditions would also seem to imply climates in which neither winters nor summers are as a rule severe. On the other hand, high temperature conditions imply climates in which the summers are long and hot and the winters short and mild. There are, of course, certain localities where extremes of both winter and summer temperatures prevail for short periods, in which case an average of moderate temperature is reasonably safe to assume for the purpose of selecting suitable consistency for a bituminous material. Another method of classifying climate which is really based upon the factors just mentioned is to divide the United States into three belts or zones—northern, middle, and southern—the general temperature conditions of which are assumed to be low, moderate, and high, respectively. It will be found, however, that it is impossible to accurately bound these three zones with any parallels of latitude, as in many individual instances a locality within one zone will have a climate similar to the general climatic conditions of one of the other zones. Everything considered, therefore, the terms low, moderate, and high may perhaps be preferable.

Terms commonly used to designate traffic conditions are just as broad as those used to denote temperature. In spite of the large amount of work on traffic classification, no satisfactory basis of classifying mixed traffic in units has as yet been devised. The terms light, moderate, and heavy are therefore commonly used. In general, light traffic signifies traffic in which the average load is relatively light and the number of vehicles passing per day is relatively small, such conditions prevail on most residential streets in cities and towns and on feeders to main State and county highways. The term moderate implies a somewhat more intense traffic than that just described, but one in which the average load is either not much heavier than that of a touring car, or if heavier does not occur in sufficient numbers to develop intense traffic. Many business streets in small towns, residential thoroughfares, park drives, main county highways, and State highways carrying principally tourist traffic fall in this class. The term heavy is used to designate both weight of average load and high intensity. Thus the principal business streets in towns and cities where trucking prevails or where the traffic is congested are said to carry heavy traffic. Many streets in shopping districts, boulevards, and main State highways also fall within this class.

ASPHALT PENETRATION LIMITS.

Based upon the factors of climate and traffic just described, the Asphalt Association has published a table of penetration limits for asphalt cements to be used in various types of highway construction. This table is offered merely as a suggestion or guide to engineers in connection with the insertion of suitable penetration limits in a specification for asphalt cement which has been recently adopted by the association. As it is probably the most detailed table which has been published, it is here presented for the purpose of inviting discussion:

Type of pavement.	Traffic.	Temperatures.		
		Low.	Moderate.	High.
Asphalt macadam	Light	120–150	90–120	80–90
	Moderate	90–120	90–120	80–90
	Heavy	80– 90	80– 90	80–90
Asphaltic concrete (coarse graded)	Light	70– 80	70– 80	60–70
	Moderate	70– 80	70– 80	60–70
	Heavy	60– 70	60– 70	60–70
Asphaltic concrete (fine graded)	Light	60– 70	60– 70	50–60
	Moderate	60– 70	60– 70	50–60
	Heavy	50– 60	50– 60	50–60
Sheet asphalt	Light	50– 60	50– 60	40–50
	Moderate	50– 60	50– 60	40–50
	Heavy	40– 50	40– 50	30–40
Asphalt block	Light	15– 25	15– 25	10–15
	Moderate	15– 25	15– 20	10–15
	Heavy	15– 20	10– 15	5–15

In connection with particular ranges of penetration, the Asphalt Association recommends that in any event a 10-point limit be allowed for asphalt cement of less than 90 penetration and that a 30-point limit be allowed for asphalt cement of over 90 penetration. Such limits may readily be met under ordinary manufacturing conditions and are sufficiently close to insure uniformity in connection with other requirements of the specifications. It will be noted that the limits apply only to the normal conditions of test, no requirements being included for penetrations at 0° C. or at 46° C. It is believed that with the materials now on the market penetration requirements at temperatures other than 25° C. are unnecessary, provided the 10 or 30 point range is adhered to as recommended. So far as actual consistency of the asphalt cement is concerned, when the temperature of the highway is 25° C., it is evident that the limits are unnecessarily close, considering the ordinary susceptibility of asphalt cements to temperature changes. Such limits are, however, advisable in order to insure that the product will not become too hard in cold weather nor too soft in hot weather. If the limits were materially increased, then requirements of penetration at either 0° C. or 46° C., or both, might be advisable, according to climatic conditions under which the material would be in use.

In connection with this table it is of interest to note that the penetrations of asphalt cement for asphalt block are very much lower than for any of the other types of pavement. This is allowable for two reasons, irrespective of the usual grading of the aggregate, which in itself would not warrant such low penetrations. In the first place, a blown type of asphalt is specified for asphalt block, a ductility requirement of from 5 to 8 being made. Such asphalts are less susceptible to temperature changes than the more ductile products specified for the other types of construction, and do not become relatively as hard in cold weather. In the second place, in asphalt block construction, prevention of contraction cracks does not have to be considered as in the case of the sheet types of construction. The necessity of using a harder asphalt cement than for the sheet types exists because of the fact that after manufacture the blocks must withstand rather rough handling during shipment and prior to laying without distortion or breakage.

TABLE SUGGESTION, NOT STANDARD.

While in general the table of limits takes into account fineness of mineral aggregate in so far as classification by types of construction is concerned, ordinary grading requirements for any one type are usually so broad that certain gradings within specification requirements may make it advisable to adopt a higher or a lower range of penetration limits for a given combination of climatic and traffic conditions than the range shown in the table. For this reason the table has been offered more in the way of suggestion than for adoption as standard. It is practically impossible to define the most suitable penetration limits for every conceivable grading which is acceptable, but fortunately experience has already demonstrated to many engineers who are obliged to use certain aggregates, such as sand from a given deposit, what penetration limits are best to use. In any event necessity of a marked departure from the limits recommended is not apt to exist.

When most of the aggregate passes the 200-mesh sieve and consists of clayey material, as in the case of asphalt-earth mixtures, the general rule that penetration of asphalt cement should decrease with fineness of the aggregate does not appear to hold true. Such an aggregate has an enormous surface area as compared with aggregates of other types of construction, and when coated with films of asphalt cement possesses such high surface friction as to create very considerable resistance to displacement. A softer asphalt cement may therefore be used than for the coarser aggregates such as sheet asphalt or Topeka. Penetrations of 90 or 100 have been successfully used under such conditions. The use of a relatively soft asphalt cement is, moreover, desirable because of the fact that asphalt-earth mixtures are not as susceptible to the kneading action of traffic and means must therefore be provided for internal adjustment to meet the tendency to crack in cold weather due to contraction of the pavement.

THE AGGREGATE AS A FACTOR.

One factor bearing upon the proper consistency of bituminous materials which was mentioned in the first part of this paper is often of considerable importance. This is the character of the aggregate. Most aggregates composed of fragments in which quartz, feldspar, and other hard minerals predominate require asphalt cements of about the penetration ranges shown in the table. When the fragments are of soft limestone, however, the use of softer asphalt cements than indicated may be proper. Thus in the construction of asphalt macadam roads with the very soft coraline rock of Florida, an asphaltic oil ordinarily suitable only for hot surface treatment may be used successfully as a binder. This would not be practicable with the harder rocks. The reason lies in the fact that the fine limestone dust which is worn off of the larger fragments amalgamates with the asphalt films to form a tough mastic of higher frictional resistance than the asphalt itself. This property, possessed also by Portland cement, is utilized in the manufacture of paving mixtures where limestone dust or Portland cement is incorporated as filler. The presence of filler toughens the mix and produces a mixture less susceptible to temperature changes.

The proper consistency of asphalt fillers for brick and block construction depends mainly upon climatic conditions and how the filler is used. For poured or squeegeed joints a partially blown type of asphalt has been adopted by the Asphalt Association with a minimum penetration requirement of 10 when tested at 0° C., under a load of 200 grams for one minute. This is to prevent the filler from becoming too brittle in cold weather. Normal penetration limits of from 40 to 50 for low temperature conditions and from 30 to 40 for high temperature conditions are suggested. In order that the filler may not become unduly soft and bleed in warm weather, a minimum melting point of 65° C. (ring and ball) is specified. A maximum melting point of 100° C. is also specified in order to insure its workability at the temperature of application and to allow sufficient time for it to flow into the joints before is congeals.

When a filler is to be applied as a grout after being mixed with hot sand, an unblown asphalt cement has been used with satisfactory result. Because of the fact that the sand materially stiffens the filler, it may be softer than the poured filler and a 20-point penetration limit at normal temperature is believed to be sufficiently close for all practical purposes. For low temperature conditions such a limit between 70 and 100 is suggested, while limits of 50 to 70 are suggested for high temperature conditions. Such ranges prevent the grout from becoming too brittle in cold weather and from bleeding in warm weather.

In order that asphalt cements may not be injuriously hardened when they are heated up to and maintained for reasonable periods at the maximum temperature of application, it has become customary to specify that the residue from the volatilization test at 163° C. shall show a penetration of not less than 50 per cent of the penetration of the original material. This is merely a precautionary requirement however as under ordinary conditions of heating in practice no asphalt cement will harden to anywhere near this extent.

SERVICE RESULTS AS GUIDES.

After all, the best guide to selection of the proper consistency of bituminous materials is experience, and it is upon service results rather than upon theoretical considerations that most of the consistency limits have been selected. Service results may, however, sometimes be misleading in this connection, and other factors than consistency should receive due consideration in attempting to analyze such results. Thus the use of too soft a bituminous material may result in the pavement shoving under traffic and also bleeding. The use of an excess of bituminous material of proper consistency may, however, cause both defects. Shoving may also be caused by lack of compaction or poor grading of the aggregate. The use of too hard a bituminous material promotes cracking of the pavement and sometimes causes it to break up or disintegrate under traffic. An insufficient quantity of material of suitable consistency, poor aggregate grading, and poor compaction may also cause these defects. Overheating or burning the bituminous material may also be responsible.

In conclusion, there appear to be at least two promising and useful lines of investigation in connection with the consistency of bituminous materials: (1) Working out a method of expressing consistencies in comparable form for all ranges from very fluid to practically solid, and (2) devising a method for accurately determining the consistency or resistance to displacement of compacted bituminous aggregates.

FEDERAL-AID ALLOWANCES

PROJECT STATEMENTS APPROVED IN FEBRUARY, 1920.

State.	Project No.	County.	Length, in miles.	Type of construction.	Project statement approved.	Estimated cost.	Federal aid.
Alabama	72	Montgomery	4.420	Gravel, surface treatment	Feb. 12	$36,342.07	$18,171.03
Arizona	21	Coconino	1.105	High class, undetermined	Feb. 24	60,258.00	25,000.00
Do	15	Gila and Graham	55.833	Earth	Feb. 27	618,604.25	309,302.12
Arkansas	53	Crittendon	13.730	Gravel	Feb. 12	107,975.89	30,000.00
Do	60	Yell	32.530	Bituminous macadam	Feb. 16	318,772.30	130,000.00
California	44	Yolo	10.710	Reinforced concrete	...do	285,021.00	142,510.50
Do	46	Humboldt	17.960	.....do	...do	424,363.50	212,181.75
Do	47	Mariposa	9.390	Earth	Feb. 19	641,927.00	198,140.00
Colorado	91	Los Animas	11.362	Gravel	Feb. 12	59,488.00	29,744.00
Do	34	Weld	2.080	Concrete	Feb. 16	74,983.31	37,491.65
Do	64	Costilla	7.888	Earth	...do	29,706.82	14,853.41
Georgia	130	Baldwin	....	Bridge	Feb 12	40,029.95	20,000.00
Do	123	Grady	....	.....do	Feb. 16	36,239.50	18,119.75
Indiana	17	Lake, Porter, and Laporte	[1] 16.800	Brick, concrete, or bituminous macadam.	...do	[1] 676,726.60	[1] 336,000.00
Do	9	Johnson	[2] 31.280	.....do	...do	[1] 1,019,409.60	[1] 509,705.00
Iowa	40	Muscatine	20.590	Earth	Feb. 12	46,604.25	23,300.00
Do	45	Mitchell	15.880	Gravel	...do	101,966.70	55,900.00
Do	47	Tama	30.000	Earth	...do	115,890.50	57,900.00
Do	53	Louisa	23.400	.....do	...do	132,682.00	66,300.00
Do	58	Benton	17.890	High class, undetermined	...do	380,939.52	190,400.00
Do	66	Kossuth	34.230	Earth	...do	121,594.00	60,700.00
Do	70	Mahaska	27.760	.....do	...do	159,060.00	79,500.00
Do	81	Taylor	19.250	.....do	...do	142,780.00	71,300.00
Do	90	Iowa	26.200	.....do	...do	123,090.00	61,500.00
Do	96	Linn	12.750	.....do	...do	45,391.50	22,600.00
Do	99	Carroll	8.880	.....do	...do	82,940.00	41,400.00
Do	103	Delaware	13.300	Gravel	...do	95,942.00	47,900.00
Do	110	Lyon	23.000	Earth	...do	69,800.50	34,900.00
Do	116	Cherokee	14.000	.....do	...do	43,912.00	21,900.00
Do	19	Grundy	14.300	.....do	Feb. 16	53,075.00	26,500.00
Kansas	45	Saline	7.000	Pavement	Feb. 14	477,433.00	105,000.00
Do	44	...do	5.250	.....do	Feb. 16	412,568.20	78,750.00
Do	47	Chase	3.790	.....do	Feb. 21	239,378.37	56,250.00
Kentucky	20	Breathitt	6.500	Earth	Feb. 12	180,627.48	90,313.74
Louisiana	27	Rapides	19.810	Gravel	Feb. 21	279,840.61	139,920.30
Do	62	Beauregard	35.930	.....do	...do	471,194.46	200,000.00
Maryland	2	Prince Georges	[1] .780	Concrete	Feb. 4	[2] 12,433.07	[2] 8,568.25
Do	6	.....do	[1] 3.380	.....do	...do	[2] 81,479.85	[2] 40,739.92
Do	7	.....do	[1] 32.121	.....do	...do	[1] 134,587.52	[1] 68,406.89
Do	8	Talbot	[1] 9.990	.....do	...do	[1] 353,774.30	[1] 176,887.15
Do	9	Frederick	[1] 3.500	.....do	...do	[1] 109,799.47	[1] 54,899.74
Do	10	Montgomery	[1] 6.175	.....do	...do	[1] 162,394.98	[1] 86,550.00
Do	13	Prince Georges	3.020	.....do	...do	[1] 98,257.30	[1] 49,128.65
Do	15	Baltimore	3.330	.....do	...do	[1] 125,409.02	[1] 62,704.51
Do	17	Harford	1.900	.....do	...do	[1] 71,308.72	[1] 35,954.36
Do	22	Caroline	2.564	.....do	...do	[1] 127,043.65	[1] 59,499.33
Do	24	Allegany	.820	.....do	...do	[1] 14,752.12	[1] 16,400.00
Do	25	Charles	4.000	Gravel	...do	[1] 89,327.81	[1] 44,663.91
Do	28	Anne Arundel	9.747	Concrete	...do	[1] 470,509.82	[1] 194,935.60
Do	29	Worcester	4.550	.....do	...do	[3] 176,000.00	[3] 88,000.00
Do	30	.....do	2.400	.....do	...do	[1] 107,917.92	[1] 53,958.96
Do	31	Charles	5.180	Gravel	...do	[1] 119,273.77	[1] 59,636.89
Do	33	Carroll	4.790	Concrete	...do	[1] 197,872.46	[1] 98,936.24
Do	34	Anne Arundel	.910	Gravel	...do	[1] 70,902.92	[1] 35,451.46
Do	35	Washington	7.49	Concrete	...do	[1] 266,542.43	[1] 133,271.22
Do	37	Kent and Queen Annes	17.000	.....do	...do	[1] 598,816.46	[1] 299,408.23
Massachusetts	31	Essex	1.107	Bituminous macadam	Feb. 16	45,544.87	22,140.00
Do	32	Hampshire	3.078	.....do	...do	172,778.37	86,389.18
Michigan	26	Antrim	7.831	Gravel	Feb. 19	119,625.00	59,812.50
Do	40	Barry	3.441	.....do	...do	57,552.00	28,776.00
Minnesota	109	Redwood	22.500	.....do	Feb. 16	44,550.00	22,275.00
Do	144	Red Lake	8.000	.....do	...do	75,680.00	37,840.00
Do	148	Hennepin and Dakota	11.670	.....do	...do	318,233.08	159,116.54
Do	124	Isanti	6.620	.....do	Feb. 19	72,377.94	36,188.97
Mississippi	26	Amite	[3] 7.860	Clay, gravel	Feb. 16[3]	[3] 22,643.50	[3] 10,000.00
Missouri	42	Barry and Lawrence	10.430	Gravel	Feb. 16	44,066.00	22,033.00
Do	74	Cedar	10.620	.....do	...do	30,744.00	15,372.00
Do	81	Jasper	2.600	Concrete	...do	79,425.50	39,712.75
Do	82	.....do	7.600	.....do	...do	31,277.40	15,638.70
Do	83	Jasper and Newton	9.500	.....do	...do	227,999.97	113,999.98
Do	84	Jasper	6.600	.....do	...do	231,137.50	115,568.75

[1] Modified agreements. Amounts given are decreases over those in the original agreements.
[2] Modified agreements. Amounts given are increases over those in the original agreements.
[3] Statements canceled or withdrawn.

PROJECT STATEMENTS APPROVED IN FEBRUARY, 1920—Continued.

State.	Project No.	County.	Length, in miles.	Type of construction.	Project statement approved.	Estimated cost.	Federal aid.
Missouri	85	Taney	4.763	Gravel	Feb. 16	$23,999.06	$11,999.53
Do	86	Hickory	4.540	do	do	25,424.00	12,712.00
Do	87	Henry	5.160	Concrete	do	125,387.70	62,693.85
Do	88	Cedar	7.480	Gravel	do	22,976.00	11,488.00
Montana	83	Valley	2.000	do	Feb. 12	13,970.00	6,985.00
Do	80	Blaine		Bridge	Feb. 18	14,999.99	7,499.99
Do	81	do		do	do	9,147.60	4,573.80
Do	69	Jefferson	.730	Pavement	Feb. 19	24,392.50	12,196.25
Nebraska	111	McPherson	11.500	Earth	Feb. 12	50,160.00	25,080.00
Do	140	Custer	11.800	do	do	37,345.00	18,672.50
Do	126	Hitchcock and Hayes	26.900	Sand-clay	Feb. 16	88,622.60	44,311.00
Do	84	Greeley	15.200	do	do	42,680.00	21,340.00
Do	128	Seward and Lancaster	29.100	Earth	Feb. 19	94,545.00	47,272.50
Do	137	Keyapaha and Rock	26.300	Sand-clay	do	148,566.00	74,283.00
Do	136	Pierce	33.300	Earth	Feb. 21	124,649.80	62,324.90
Do	91	Washington	[1] 3.380	do	Feb. 19		
New Mexico	43	Torrence	17.000	do	Feb. 16	52,360.00	26,180.00
Do	41	Otero	18.000	Gravel	do	57,710.12	28,855.06
Do	45	Luna	5.380	do	Feb. 19	32,892.75	16,446.37
New York	46	Dutchess	2.500	Reinforced concrete	Feb. 21	100,000.00	50,000.00
North Carolina	117	Wilson	6.200	Sand-clay	Feb. 19	48,796.00	24,398.00
Do	119	Bladen	23.000	do	do	164,153.00	82,076.50
Do	120	do	23.000	do	do	157,740.00	78,870.00
Do	115	Henderson	10.000	do	Feb. 24	100,903.20	50,451.60
Do	124	Surry	18.000	do	do	141,644.80	70,822.40
Do	86	Martin and Bertie	3.900	do	Feb. 21	427,072.80	213,536.40
North Dakota	104	Hettinger		Bridge	Feb. 16	15,400.00	7,700.00
Do	105	do		do	do	19,800.00	9,900.00
Oklahoma	25	Kingfisher	1.485	Pavement	Feb. 12	59,424.79	29,712.39
Do	28	Woods and Woodward		Bridge	Feb. 14	152,000.00	76,000.00
Do	26	Grant		do	Feb. 19	121,528.00	60,764.00
South Carolina	73	Spartanburg	12.636	Sand-clay	do	151,189.89	50,000.00
Do	84	Laurens	14.131	do	do	84,863.19	25,000.00
Do	36	Chesterfield	11.346	do	Feb. 21	85,358.47	24,000.00
South Dakota	47	Lincoln	15.000	Gravel	Feb. 16	122,200.10	61,100.05
Do	41	Marshall and Roberts	24.400	do	Feb. 19	178,476.10	89,238.05
Do	44	Roberts	10.400	do	do	80,173.50	40,086.75
Do	48	Moody	3.850	do	do	34,729.20	17,364.60
Do	50	Sanborn	12.000	do	Feb. 21	104,060.00	52,030.00
Do	37	Faulk and Potter	13.000	Earth	do	70,697.00	35,348.50
Do	38	Jerauld	16.800	Gravel	do	104,995.00	52,497.50
Do	52	Minnehaha	10.250	do	do	96,071.80	48,035.90
Do	53	Miner and Lake	11.000	do	do	85,013.50	42,506.75
Do	45	Grant	13.900	do	do	141,267.50	70,633.75
Do	26	Kingsbury	[2] .590	do	do	[1] 970.20	[1] 485.10
Tennessee	30	Tipton	5.152	Bituminous macadam	do	148,305.74	74,152.87
Do	42	Hamblen	8.314	do	do	180,355.11	90,177.55
Texas	135	Schleicher	30.000	Water-bound macadam	Feb. 12	194,999.97	85,000.00
Do	141	Polk	20.000	Gravel	do	153,395.00	38,348.75
Do	134	do	40.000	Gravel treatment	Feb. 14	609,257.00	152,314.25
Do	133	Kimble	34.900	Gravel	Feb. 19	200,184.72	100,000.00
Do	144	Nacogdoches	18.640	do	do	233,774.47	58,443.61
Do	147	Smith	24.600	do	do	244,915.00	112,742.76
Do	146	Delta	11.310	do	Feb. 21	306,500.50	95,000.00
Do	148	Klebing	19.640	do	do	429,999.89	125,000.00
Do	1	Culberson	[3] 50.400	Earth	Feb. 27	[3] 45,265.55	[3] 22,632.77
Do	47	Collingsworth	[3] 10.500	Sand-clay	Feb. 6	[3] 10,010.00	[3] 5,000.00
Utah	11	Iron	33.300	Gravel	Feb. 16	331,328.02	165,664.01
Vermont	14	Windham	5.600	do	do	122,003.20	61,001.60
Virginia	74	Princess Anne	10.880	Concrete	do	364,870.00	182,435.00
Washington	50	Spokane	4.110	do	Feb. 21	159,921.30	79,960.65
West Virginia	77	Greenbrier	6.750	Bituminous macadam	Feb. 16	66,500.00	29,191.40
Wisconsin	173	Rusk	1.020	Earth	Feb. 14	9,326.37	3,300.96
Do	161	Juneau	0.935	Concrete	Feb. 16	29,986.66	10,086.83
Do	171	Richland		Bridge	do	29,581.20	10,427.07
Do	126	St. Croix	2.750	Concrete	Feb. 19	92,560.94	33,360.00
Do	108	Juneau	1.630	Earth	Feb. 21	27,891.78	11,547.00
Do	111	Eau Claire	1.890	Reinforced concrete	do	75,753.15	28,253.00
Do	172	Richland	3.500	Earth	do	50,400.00	18,000.00
Wyoming	40	Niobrara	11.894	do	Feb. 10	35,640.00	17,820.00
Do	55	Uinta	4.012	Gravel	Feb. 16	27,445.00	13,722.50
Do	56	Converse	7.224	Sand-clay	do	38,874.00	19,437.00
Do	59	Carbon	28.608	Earth	do	103,015.00	51,507.50
Do	60	do	2.384	Sand-clay	do	11,110.00	5,555.00
Do	49	Sweetwater	43.920	Earth	Feb. 24	164,670.00	82,335.00
Do	46	Laramie	25.304	Sand-clay	Feb. 21	93,280.00	46,640.00
Do	13	Carbon	[3] 13.880	Earth	Feb. 7	[3] 14,500.75	[3] 7,250.37

[1] Revised statements. Amounts given are decreases over those in the original statements.
[2] Modified statements. Amounts given are increases over those in the original statements.
[3] Statements canceled or withdrawn.

PROJECT AGREEMENTS EXECUTED IN FEBRUARY, 1920.

State.	Project No.	County.	Length, in miles.	Type of construction.	Project agreement signed.	Estimated cost.	Federal aid.
Alabama	57	Jefferson	5.670	Sheet asphalt	Feb. 19	$255,085.42	$113,400.00
Do	61	Crenshaw	11.113	Sand-clay	...do...	39,257.84	19,252.29
Do	67	Dale	22.290	.....do	...do...	139,800.66	69,900.33
Do	65	Pike	8.810	.....do	Feb. 2	52,853.84	26,426.91
Do	38	Lawrence		Gravel	Feb. 14	[1] 10,838.08	[1] 5,419.04
Do	37	Jackson		.....do	Feb. 16	[1] 1,814.26	[1] 907.13
Do	17	Greene		.....do	Feb. 10	[1] 35,622.86	[1] 17,811.43
Arizona	2	Maricopa		Concrete	Feb. 19	[1] 1,362.90	[1] 7,971.41
California	35	San Diego	13.800	Earth	Feb. 18	224,649.11	112,324.55
Do	33	San Bernardino and Riverside	7.100	.....do	...do...	107 054.79	53,527.39
Do	34	San Diego	6.610	.....do	...do...	78,399.04	39,199.52
Do	32	Siskiyou	17.640	Concrete	Feb. 26	422,063.13	211,031.56
Colorado	62	Otero	0.833	.....do	Feb. 19	29,011.40	14,505.70
Do	63	.....do	0.843	.....do	...do...	27,905.35	13,952.67
Georgia	53	Bibb	2.140	.....do	Feb. 6	74,644.02	37,322.01
Do	46	.....do	8.700	.....do	Feb. 17	263,645.58	131,822.79
Do	33	Troup		Sand-clay	Jan. 29	[1] 20,985.31	[1] 8,000.00
Do	6	Hall and Lumpkin	[1] 0.352	Gravel	Feb. 10	[1] 5,691.82	[1] 2,845.91
Illinois	8C	Sangamon, Madison, Macoupin	5.684	Concrete	...do...	190,563.91	95,281.95
Do	8H	.....do	5.678	.....do	...do...	211,965.26	105,982.63
Do	8P15d	.....do	4.641	.....do	...do...	167,982.94	83,991.47
Do	8Q	.....do	2.749	.....do	...do...	111,926.49	54,980.00
Do	8S	.....do	4.813	.....do	...do...	199,598.82	96,260.00
Do	8U	.....do	3.749	.....do	...do...	148,020.90	74,010.45
Do	W.15d	.....do	5.033	.....do	...do...	168,357.27	84,178.63
Do	8Y	.....do	4.029	.....do	...do...	175,006.27	82,416.09
Iowa	104	Polk	6.570	Pavement	Feb. 24	339,418.20	131,000.00
Do	39	Green	7.410	.....do	...do...	302,539.82	148,200.00
Maine	3	Kennebec		Bituminous macadam	Feb. 10	[1] 75,189.15	[1] 73,945.22
Maryland	28A	Anne Arundle	0.753	Concrete	...do...	35,490.18	15,064.40
Do	31	Charles	2.020	Gravel	...do...	23,726.23	11,863.11
Do	35	Washington	2.010	Sheet asphalt	Feb. 19	65,107.57	32,553.78
Do	32	St. Marys	5.850	Gravel	...do...	76,882.30	38,441.15
Do	30	Worcester	1.900	Concrete	...do...	61,482.08	30,741.04
Do	13B	Prince Georges	0.830	.....do	Jan. 31	38,066.60	19,033.30
Do	13A	.....do	0.190	.....do	Feb. 12	[2] 6,264.44	[2] 3,132.22
Massachusetts	27	Essex	1.766	Bituminous, concrete, and macadam	Feb. 19	45,512.17	22,756.08
Do	5	Berkshire		Water-bound macadam	Jan. 19	[1] 40,247.35	[1] 26,816.30
Michigan	32	Van Buren and Allegan	7.039	Concrete	Feb. 11	218,600.47	109,300.23
Do	18	Grand Traverse	3.113	Gravel	Feb. 16	45,978.58	22,989.29
Minnesota	24	Morrison	11.950	.....do	Feb. 10	50,195.84	25,000.00
Do	31	Kandiyohi	0.730	Earth and concrete	Feb. 19	[1] 248,281.47	[1] 125,000.00
Mississippi	86	Lincoln	3.610	Gravel	...do...	61,214.54	30,607.27
Montana	40	Gallatin	2.840	.....do	Feb. 18	23,017.28	11,508.64
Do	8	Meagher	9.830	Earth	Feb. 19	29,437.32	14,718.66
Do	53A	Yellowstone	26.040	Gravel	...do...	121,325.08	60,662.54
Do	9A	Madison	5.500	Earth	...do...	16,098.00	8,049.00
Do	3	Carbon		Gravel	...do...	[1] 2,999.14	[1] 1,499.57
Nebraska	50AB	Merrick and Nance	19.700	Sand-clay	Feb. 16	88,226.23	44,113.11
Do	100	Douglas	10.290	Earth	Feb. 3	60,379.85	30,189.92
Do	67A	Red Willow	4.560	.....do	Feb. 18	25,724.79	12,862.39
Do	16	Kimball and Banner	26.640	.....do	Feb. 19	101,883.50	50,941.75
Do	78	Douglas	12.830	.....do	Feb. 16	88,142.25	44,071.12
Nevada	25	White Pine	.280	.....do	Feb. 19	8,255.22	4,127.61
Do	1	Humboldt		Gravel	...do...	[1] 42,516.82	[1] 21,258.41
New Hampshire	51	Cheshire		Bituminous macadam	Feb. 4	[1] 1,079.76	[1] 539.88
New Jersey	18	Mercer	1.648	Concrete	Feb. 18	85,750.44	32,960.00
North Carolina	29	Union	8.660	Sand-clay	Feb. 11	49,535.34	24,767.67
Do	67A	Nash	5.000	Pavement	...do...	202,768.01	101,384.00
Do	54	Wake	6.811	Concrete	Feb. 19	249,585.19	124,792.59
Do	34	Wayne		Bridge	Feb. 10	39,982.73	10,000.00
Do	25	Person		Sand-clay	...do...	[1] 16,105.76	[1] 8,052.88
North Dakota	38	Hettinger	8.880	Earth	Feb. 19	29,374.99	14,687.49
Do	20	Wells		.....do	Feb. 16	[1] 8,620.33	[1] 4,310.16
Ohio	34	Erie	3.051	Reinforced concrete	Feb. 19	113,000.00	30,000.00
Do	35	Portage	8.483	Water-bound macadam	...do...	224,000.00	84,700.00
Do	86	Paulding	2.019	Concrete	Feb. 27	61,700.00	20,000.00
Do	96	.....do	2.023	.....do	...do...	62,000.00	20,000.00
Oregon	29	Coos and Douglas	14.170	Earth	Feb. 25	387,301.99	193,650.99
South Carolina	23	Greenville	3.030	Bituminous macadam	Feb. 10	52,137.92	26,068.96
Do	33	Beaufort		Bridge	Jan. 28	20,759.20	10,379.60
South Dakota	2	Lincoln	9.970	Gravel	Feb. 19	71,842.03	35,921.01
Do	22	Day	28.070	.....do	...do...	237,379.51	118,689.75
Do	24	Hanson	27.363	.....do	...do...	173,657.83	86,828.91
Tennessee	15	Madison		Bridge	Feb. 17	22,797.39	11,398.69
Texas	68	Gonzales	27.710	Gravel	Feb. 19	233,053.84	79,473.31
Virginia	39	Buckingham	6.264	Sand-clay	Feb. 18	57,036.43	28,518.21
Do	33	Nelson		Water-bound macadam	...do...	[1] 3,172.40	[1] 1,586.20
Washington	18	Skagit	5.580	Earth	Feb. 19	66,142.62	33,071.31
Do	46	Garfield	10.810	Gravel	...do...	202,402.72	101,201.36
Do	48	Ferry	1.190	Earth	...do...	23,639.77	11,819.88
Do	47	Columbia	6.570	Crushed rock	Feb. 24	104,702.64	52,351.32
West Virginia	26	Lincoln	2.360	Concrete	Feb. 2	52,899.00	26,449.50
Do	44	Mercer	4.710	Water-bound macadam	...do...	122,961.79	61,480.89
Do	48	Boone	5.023	Earth	Feb. 19	79,900.00	33,000.00
Do	51	Wayne		.....do	...do...	[1] 22,626.45	[1] 11,313.15
Wisconsin	78	Pepin	1.493	Gravel	...do...	11,683.59	3,894.53
Do	16	.....do	2.340	.....do	...do...	25,156.76	8,385.59
Do	2	Manitowoc	[1] .560	Concrete	Feb. 12	[1] 20,064.62	[1] 6,688.21

[1] Modified agreements. Amounts given are increases over those in the original agreements.
[2] Modified agreements. Amounts given are decreases over those in the original agreements.

TESTS OF ROAD-BUILDING ROCK IN 1919

CONTINUING the practice inaugurated last year, Public Roads publishes this month the physical tests of road-building rock made in the laboratories of the Bureau of Public Roads from January 1, 1919, to January 1, 1920.

The results of all tests made up to January 1, 1916, were published in Department of Agriculture Bulletin No. 370, entitled "The Results of Physical Tests of Road-Building Rock;" those made during 1916 and 1917 were reported in Bulletin No. 670; volume 1, No. 11, of Public Roads, issued in March, 1919, contained the results of the 1918 tests; and the following tables contain the test data on samples received since that time up to January 1, 1920.

Results of physical tests of road-building rock from the United States and Canada, Jan. 1, 1919, to Jan. 1, 1920.

Serial No.	Town or city.	State and county.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot, pounds.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
		ALABAMA.								
14450	(1)	Calhoun	Quartzite schist	(2)	164	0.38	4.4	9.1	18.7	19
		ARIZONA.								
14119	Douglas	Cochise	Copper slag	(2)	217	.58	5.5	7.3	(2)	9
15296	(1)	Coconino	Calcareous sandstone	(2)	134	8.41	23.3	1.7	.0	2
15297	(1)	do	Siliceous limestone	(2)	153	4.20	7.9	5.1	18.7	6
15298	Grand Canyon	do	Chert	(2)	120	4.50	13.7	2.9	(2)	(2)
15299	(1)	do	Dolomite	(2)	162	2.28	5.0	8.0	15.0	6
13977	Globe	Gila	Copper slag	(2)	221	1.28	3.2	12.5	(2)	(2)
13978	do	do	do	(2)	220	1.43	3.1	12.9	(2)	(2)
14746	do	do	do	(2)	216	1.75	6.8	5.9	15.0	9
15190	Clifton	Greenlee	do	(2)	207	.37	4.5	8.9	18.0	27
		ARKANSAS.								
15386	(1)	Franklin	Sandstone	(2)	158	2.26	5.1	7.8	17.0	11
14425	(1)	Independence	Crystalline limestone	(2)	166	.60	10.4	3.8	8.3	3
14883	Penters Bluff	do	Chert	(2)	157	2.04	10.8	3.7	(2)	(2)
14885	do	do	Limestone	(2)	166	.67	5.4	7.4	15.3	7
14886	do	do	do	(2)	167	.69	10.5	3.8	7.3	3
14665	Lamar	Johnson	Feldspathic sandstone	(2)	156	2.51	3.5	11.4	(2)	(2)
14959	do	do	do	(2)	(2)	(2)	(2)	(2)	18.0	14
14279	Little Rock	Pulaski	Syenite	(2)	156	1.23	5.0	8.0	18.7	8
14280	16 miles west of Little Rock.	do	Sandstone	(2)	157	1.76	6.4	6.3	16.3	10
14293	Little Rock	do	Porphyritic syenite	(2)	163	.20	3.7	10.8	18.5	12
		CALIFORNIA.								
14496	Kennett	Shasta	Copper slag	(2)	221	1.09	4.9	8.2	17.7	9
		COLORADO.								
13892	Near Boulder	Boulder	Feldspar basalt	(2)	174	.56	2.9	13.8	18.8	18
14998	Denver	Denver	Lead slag	(2)	217	.49	7.8	5.1	(2)	(2)
15016	Pueblo	Pueblo	Lead smelter slag	(2)	213	.53	5.5	7.3	(2)	(2)
15017	do	do	do	(2)	223	.44	6.0	6.7	(2)	(2)
15018	do	do	do	(2)	220	.34	5.6	7.1	(2)	(2)
		CONNECTICUT.								
14965	(1)	Hartford	Altered diabase	(2)	184	.25	3.0	13.3	18.0	20
14970	(1)	do	do	(2)	185	.24	2.7	14.8	17.3	22
13865	Oneco	Windham	Granite	(2)	(2)	(2)	4.5	8.9	18.0	8
		DELAWARE.								
15401	(1)	Kent	Pyroxene quartzite	(2)	177	.18	3.3	12.1	18.7	13
13893	Elsmere	New Castle	Plagioclase gneiss	(2)	168	.20	3.6	11.1	18.7	9
14025	Stoney Battery	do	Hornblende schist	(2)	191	.25	3.6	11.1	17.7	8
14114	Wilmington	do	Hornblende gneiss	(2)	194	.16	3.1	12.9	17.2	11
15367	do	do	Pyroxene quartzite	(2)	175	.69	3.3	12.1	18.7	12
		GEORGIA.								
13914	Atlanta	Fulton	Granite	(2)	(2)	(2)	4.4	9.1	18.0	9
15378	Near Talbot	Talbotton	Diabase	(2)	187	.32	1.7	23.5	18.7	27
		ILLINOIS.								
14426	South Chicago	Cook	Slag	(2)	141	3.48	9.1	4.4	14.3	6
14949	(1)	Madison	Fossiliferous limestone	(2)	166	1.19	6.0	6.7	18.3	10
		INDIANA.								
15352	Henryville	Clark	Dolomitic limestone	(2)	172	1.06	3.5	11.4	17.0	13
14871	Eaton	Delaware	Dolomite	(2)	169	1.43	5.8	6.9	15.7	6
15374	Floyds Knobs	Floyd	Limestone	(2)	164	2.43	4.7	8.5	13.3	6
15375	do	do	Feldspathic sandstone	(2)	132	9.64	11.8	3.4	3.7	4
15376	do	do	do	(2)	134	9.75	10.9	3.7	0.0	3
14730	Monon	White	Dolomite	(2)	170	.41	5.6	7.1	15.7	7
		IOWA.								
13913	Brandon	Blackhawk	Limestone	(2)	161	.36	4.7	8.5	15.0	4
13903	Stone City	Jones	Dolomite	(2)	132	11.81	14.5	2.7	0.0	4
		KANSAS.								
14882	2½ miles west Woodruff	Phillips	Sandstone	(2)	144	1.77	5.7	7.0	18.7	7
14711	(1)	Riley	Dolomite	(2)	174	2.20	4.0	10.0	15.3	10

[1] Exact locality not known.

[2] Test not made.

Results of physical tests of road-building rocks from the United States and Canada, Jan. 1, 1919, to Jan. 1, 1920—Continued.

Serial No.	Town or city.	State and county.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot, pounds.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
		MAINE.								
13862	[1]	Hancock	Granite	[2]	[2]	[2]	3.7	10.8	18.7	10
13866	[1]	Knox	do	[2]	[2]	[2]	3.4	11.8	18.0	10
13868	[1]	do	do	[2]	[2]	[2]	3.4	11.8	18.7	10
13869	[1]	do	do	[2]	[2]	[2]	3.3	12.1	18.7	10
		MARYLAND.								
14167	Baltimore	Baltimore	Copper blast-furnace slag	[2]	203	.97	5.5	7.3	17.3	13
14179	do	do	do	[2]	205	.95	5.2	7.7	15.3	11
14193	do	do	do	[2]	199	.72	3.1	12.9	[2]	[2]
14646	Towson	do	Marble	[2]	175	.38	9.9	4.1	11.0	4
15387	[1]	do	do	[2]	172	.30	5.4	7.4	[2]	[2]
15161	Sylmar-Rising Sun	Cecil	Hypersthene granite	[2]	183	2.02	[2]	[2]	18.0	10
14548	Bel Air	Harford	Amphibolite	[2]	190	.17	2.8	14.3	18.7	21
14424	Weaverton	Washington	Quartzite	[2]	167	.31	3.1	12.9	[2]	[2]
		MASSACHUSETTS.								
15178	Seekonk	Bristol	Feldspathic sandstone	[2]	165	1.21	4.3	9.3	18.7	14
15179	do	do	Siliceous slate	[2]	164	1.84	3.9	10.3	[2]	[2]
14987	[1]	Essex	Trachytic rhyolite	[2]	166	.10	3.1	12.9	19.3	15
15174	Rockport	do	Granite	34,200	[2]	[2]	[2]	[2]	[2]	11
15175	do	do	do	31,225	[2]	[2]	[2]	[2]	[2]	10
15176	do	do	do	24,275	[2]	[2]	[2]	[2]	[2]	8
15177	do	do	do	32,125	[2]	[2]	[2]	[2]	[2]	11
15156	Cummington	Hampshire	Mica-schist	[2]	166	1.54	6.2	6.5	17.0	9
15157	do	do	Chlorite mica-schist	[2]	165	1.35	7.7	5.2	17.3	6
15158	do	do	Mica-schist	[2]	131	.55	4.3	9.3	17.7	7
14890	Chelmsford	Middlesex	Muscovite granite	[2]	163	.88	3.9	10.3	18.7	9
13885	Westford	do	Granite	[2]	[2]	[2]	4.6	8.7	18.0	9
13894	do	do	do	[2]	[2]	[2]	3.1	12.9	19.3	9
14652	Quincy	Norfolk	Altered granite	[2]	163	.91	5.3	7.6	18.7	[2]
14897	Randolph	do	Feldspathic quartzite	[2]	163	.91	3.3	12.1	18.7	10
15292	Hingham	Plymouth	Aplitic granite	[2]	130	.42	2.7	14.8	18.3	18
15291	[1]	Suffolk	Altered rhyolite	[2]	131	.59	2.6	15.4	19.0	12
14878	West Roxbury	do	Altered diabase	[2]	192	.19	1.8	22.2	18.7	24
14917	do	do	Granite	[2]	167	.25	4.1	9.8	18.7	10
15164	[1]	do	Altered diabase	[2]	192	.63	2.2	18.2	18.0	18
		MICHIGAN.								
14623	Rogers	Presque Isle	Limestone	[2]	162	1.30	9.5	4.2	13.3	5
14689	Inwood	Schoolcraft	Dolomitic marble	[2]	177	.46	4.8	8.3	16.0	8
		MINNESOTA.								
14495	[1]	Rock	Quartzite	[2]	168	.38	2.4	16.7	[2]	[2]
15214	[1]	St. Louis	Epidosite	[2]	218	.48	2.6	15.4	18.7	20
		MISSISSIPPI.								
14283	Iuka	Tishomingo	Weathered chert	[2]	127	7.89	12.2	3.3	[2]	[2]
		MISSOURI.								
14731	Kansas City	Jackson	Limestone	[2]	164	.74	5.5	7.3	13.3	5
14448	North of Webb City	Jasper	Chert	[2]	155	2.96	4.4	9.1	19.7	[2]
		MONTANA.								
14363	Logan	Gallatin	Limestone	[2]	169	.36	3.4	11.8	[2]	[2]
14466	Trident	do	do	[2]	166	.72	6.8	5.9	[2]	[2]
14467	do	do	Siliceous limestone	[2]	164	1.43	3.4	11.8	17.0	14
14360	Livingston	Park	Basalt	[2]	178	1.95	4.6	8.7	16.7	10
13897	[1]	Yellowstone	Sandstone	[2]	135	9.27	26.3	1.5	13.5	7
		NEBRASKA.								
14581	[1]	Cass	Cherty limestone	[2]	163	1.27	4.8	8.3	15.0	7
14582	[1]	do	Siliceous limestone	[2]	162	2.65	6.7	6.0	13.3	5
		NEW HAMPSHIRE.								
13864	[1]	Cheshire	Granite	[2]	[2]	[2]	4.0	10.0	16.7	8
13860	[1]	Hillsboro	do	[2]	[2]	[2]	5.1	7.8	18.0	7
13861	[1]	do	do	[2]	[2]	[2]	4.3	9.3	17.3	8
13883	Brookline	do	do	[2]	[2]	[2]	3.3	12.1	18.0	8
13863	[1]	Merrimack	do	[2]	[2]	[2]	3.4	11.8	18.7	9
13867	[1]	do	do	[2]	[2]	[2]	3.4	11.8	18.7	10
14892	Concord	do	do	[2]	164	.75	4.6	8.7	17.7	10
		NEW JERSEY.								
14638	[1]	Essex	Basalt	[2]	183	.15	2.4	16.4	17.6	12
14869	[1]	Hunterdon	Dolomite	[2]	177	.39	3.0	13.3	15.3	8
14379	Chrome	Middlesex	Copper slag	[2]	221	.45	4.1	9.8	16.7	25
14644	Maurer	do	do	[2]	218	1.35	2.9	13.8	16.7	15
14973	Chimney Rock	Somerset	Basalt	[2]	180	.59	2.1	19.0	18.0	18
14641	Bound Brook	do	Diabase	[2]	186	.15	1.6	25.0	19.3	27
14982	[1]	do	Altered basalt	[2]	183	.29	2.2	18.2	18.7	21
14983	[1]	do	do	[2]	186	.21	1.8	22.2	18.0	31
		NEW YORK.								
13890	Alexandria Bay	Jefferson	Granite	[2]	[2]	[2]	2.9	13.8	18.7	11
14462	Oaks Corners	Ontario	Argillaceous limestone	[2]	166	1.63	3.8	10.5	14.3	8

[1] Exact lacality nat known.

[2] Test not made.

Results of physical tests of road-building rocks from the United States and Canada, Jan. 1, 1919, to Jan. 1, 1920—Continued.

Serial No.	Town or city.	State and county.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot, pounds.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
		NORTH CAROLINA.								
13901	Craggy	Buncombe	Granite	(2)	163	0.49	2.8	14.3	18.5	12
14753	(1)	Durham	Gneissoid granite	(2)	165	.43	4.1	9.8	18.0	7
14403	Belmont	Gaston	Biotite granite	(2)	171	.39	3.5	11.4	17.8	8
14514	(1)	Mecklenberg	Weathered granite	(2)	163	.37	4.5	8.9	18.0	8
14515	(1)	do	Quartzite	(2)	166	.54	2.3	17.4	(2)	(2)
14516	(1)	do	Uralite diabase	(2)	182	.80	2.9	13.8	18.7	14
15285	(1)	do	Altered biotite granite	(2)	134	.65	3.7	10.8	18.3	26
14633	(1)	do	Epidote quartzite	(2)	174	.85	3.0	13.3	18.7	29
13972	Salisbury	Rowan	Granite	(2)	(2)	(2)	3.3	12.1	19.0	10
15009	Granite Quarry	do	do	(2)	163	.57	3.5	11.4	18.0	15
14891	Mount Airy	Surry	do	(2)	164	.71	4.6	8.7	18.0	6
14244	Henderson	Vance	Gneissoid granite	(2)	170	.45	2.8	14.3	18.0	6
15078	Greystone	do	do	35,140	163	.72	3.6	11.1	18.0	7
15079	do	do	do	31,050	162	.79	3.4	11.8	18.0	7
14174	Wendell	Wake	Biotite granite	(2)	(2)	(2)	(2)	(2)	18.0	5
14175	Raleigh	do	do	(2)	163	.47	3.2	12.5	(2)	(2)
15012	Near Raleigh	do	Olivine diabase	(2)	186	.38	1.7	23.5	18.0	19
		OHIO.								
14016	Delphos	Allen	Argillaceous dolomite	(2)	169	.20	4.8	8.3	16.2	6
14018	do	do	Dolomite	(2)	174	1.04	5.1	7.8	15.3	9
14019	do	do	do	(2)	171	.89	4.4	9.1	14.7	9
14772	do	do	Argillaceous dolomite	(2)	170	.70	4.8	8.3	15.3	13
15216	Lima	do	do	(2)	165	1.86	3.7	10.8	14.7	9
14524	Richland	Clinton	do	(2)	157	3.48	9.4	4.3	13.3	5
15096	Latonia	Columbiana	Blast-furnace slag	(2)	111	8.42	16.2	2.5	(2)	(2)
15215	Delaware	Delaware	Argillaceous limestone	(2)	163	.93	5.3	7.6	15.8	8
14525	Homer	Highland	Argillaceous dolomite	(2)	169	1.35	7.4	5.4	12.7	5
14833	Lorain	Lorain	Blast furnace slag	(2)	123	5.89	16.8	2.4	(2)	(2)
15217	Circleville	Pickaway	Dolomitic marble	(2)	164	2.82	4.4	9.1	14.7	9
13888	Woodville	Sandusky	Dolomite	(2)	159	3.85	5.8	6.9	16.0	7
14771	Middlepoint	Van Wert	do	(2)	166	2.07	5.0	7.9	15.7	8
15047	North Baltimore	Wood	Argillaceous dolomite	(2)	168	2.57	4.6	8.7	(2)	(2)
		OKLAHOMA.								
14997	Near Armstrong	Bryan	Fossiliferous limestone	(2)	164	1.49	6.3	6.4	13.3	6
15118	Near Caddo	do	Limestone	(2)	153	5.07	6.4	6.3	0.0	5
14914	Tulsa	Tulsa	Argillaceous limestone	(2)	165	1.16	7.1	5.6	14.7	6
		PENNSYLVANIA.								
14382	Carnegie	Allegheny	Copper slag	(2)	238	.38	4.3	9.3	17.3	14
14468	Birdsboro	Berks	Diabase	(2)	188	.35	(2)	(2)	18.7	10
14642	Douglass Township	do	Altered slate	(2)	170	.32	1.9	21.1	18.7	28
14150	Johnstown	Cambria	Open-hearth slag	(2)	186	2.02	11.0	3.6	(2)	(2)
14151	do	do	Slag	(2)	128	6.88	18.9	2.1	(2)	(2)
14111	Palmerton	Carbon	Spiegel slag	(2)	122	7.52	13.1	3.1	(2)	(2)
14294	Howellville	Chester	Marble	(2)	173	.36	4.3	9.3	(2)	(2)
14943	Devault	do	Dolomitic marble	(2)	176	.70	7.7	5.2	14.0	5
13993	Point Marion	Fayette	Feldspathic sandstone	(2)	151	3.53	10.3	3.9	11.0	6
15199	Fayetteville	Franklin	Quartzite	(2)	158	.85	4.9	8.2	19.0	11
14999	Emaus	Lehigh	Altered diorite	(2)	175	.77	5.8	6.9	(2)	7
15351	(1)	do	Diorite	(2)	184	.91	5.0	8.0	18.0	10
13939	Ivy Rock	Montgomery	Dolomitic marble	(2)	177	.26	4.9	8.2	15.3	7
14691	Gwynedd Valley	do	Siliceous limestone	(2)	169	.35	2.3	17.4	17.7	19
15264	Glendon	Northampton	Slag	(2)	174	1.98	6.1	6.6	15.7	8
13907	Holmesburg	Philadelphia	Granite	(2)	(2)	(2)	4.9	8.2	18.7	8
14785	Matamoras	Pike	Feldspathic sandstone	(2)	165	1.04	5.0	8.0	15.0	10
		SOUTH CAROLINA.								
14765	Near Rion	Fairfield	Biotite granite	20,100	164	.69	3.5	11.4	18.0	12
14457	Cayce	Lexington	do	33,090	163	.51	2.5	16.0	18.3	13
14298	4 miles east of McCormick.	McCormick	Altered andesite	(2)	171	2.82	10.6	3.8	16.7	8
14301	Near McCormick	do	Quartz	(2)	163	.34	9.8	4.1	(2)	(2)
15153	Lockhart	Union	Diorite	(2)	189	.64	2.4	16.7	19.0	10
		TENNESSEE.								
14464	Johnson City	Washington	Siliceous limestone	(2)	175	.29	2.5	16.0	17.3	16
14840	Brayesville	do	Limestone	(2)	170	.84	6.6	6.1	15.3	4
14841	Washington College	do	do	(2)	169	.49	5.7	7.0	15.3	5
		TEXAS.								
13980	El Paso	El Paso	Lead slag	(2)	222	1.60	7.1	5.6	(2)	(2)
15004	(1)	do	Smelter slag	(2)	224	.80	4.8	8.3	(2)	(2)
15005	(1)	do	Lead smelter slag	(2)	212	.65	9.8	4.1	(2)	(2)
		UTAH.								
15261	Poison Spring Bench	Price	Argillaceous limestone	(2)	139	4.66	8.4	4.8	10.7	5
14250	Price	Carbon	Limestone	(2)	168	.47	4.9	8.2	16.7	6
14251	do	do	Argillaceous limestone	(2)	166	.91	4.5	8.9	15.3	7
14257	do	do	do	(2)	156	3.28	5.8	6.9	15.3	7
14258	do	do	Calcareous sandstone	(2)	161	2.15	6.3	6.4	14.7	8
14867	(1)	Carbon and Emery	Argillaceous limestone	(2)	144	5.93	7.3	5.5	12.0	3
14868	(1)	do	Limestone	(2)	166	.87	4.7	8.5	16.0	7
14252	Castledale	Emery	Limestone conglomerate	(2)	154	3.95	17.3	2.3	(2)	(2)
14253	do	do	Argillaceous limestone	(2)	168	.48	4.7	8.5	16.0	6
14254	do	do	Calcareous sandstone	(2)	161	2.08	3.3	12.1	15.3	19
14255	do	do	do	(2)	166	1.79	7.6	5.3	14.7	7
14249	Emery	do	Limestone	(2)	162	2.16	7.1	5.6	16.0	12
13999	Garfield	Salt Lake	Copper slag	(2)	210	1.03	4.9	8.2	(2)	(2)
15107	do	do	do	(2)	202	.27	5.0	8.0	(2)	(2)
15109	do	do	do	(2)	208	1.37	6.6	6.0	18.0	18
14643	Midvale	do	Lead slag	(2)	221	.97	4.7	8.5	14.7	7

1 Exact locality not known.

2 Test not made.

Results of physical tests of road-building rocks from the United States and Canada, Jan. 1, 1919, to Jan. 1, 1920—Continued.

Serial No.	Town or city.	State and county.	Name of material.	Crushing strength, pounds per square inch.	Weight per cubic foot, pounds.	Absorption, pounds per cubic foot.	Per cent of wear.	French coefficient of wear.	Hardness.	Toughness.
		VIRGINIA.								
14131	Meachums River	Albemarle	Biotite gneiss	(2)	177	0.74	5.4	7.4	17.0	6
14870	Lynchburg	Amherst	Amphibolite	(2)	188	.81	5.3	7.6	16.7	7
14881	Lone Fountain	Augusta	Limestone	(2)	169	.22	5.8	6.9	16.0	4
15187	(1)	do	do	(2)	169	.51	6.3	6.4	15.7	4
15188	(1)	do	do	(2)	168	.52	5.0	8.0	15.7	9
13906	Blue Ridge	Bedford	Siliceous dolomite	(2)	174	.17	4.8	8.3	16.7	10
13990	West of Lynchburg	do	Gabbro	(2)	175	.31	4.9	8.2	18.7	7
14224	Major	do	Altered andesite	(2)	188	.43	2.7	14.8	17.3	9
14449	(1)	Botetourt	Limestone	(2)	195	.17	6.2	6.5	15.0	5
15111	Near Buchanan	do	Siliceous dolomite	(2)	176	.65	3.3	12.1	16.3	12
14651	Near Bocock	Campbell	Sericite gneiss	(2)	165	.77	6.0	6.7	18.0	7
15011	Herndon	Fairfax	Rhyolite breccia	(2)	167	2.32	5.9	6.8	(2)	(2)
14460	(1)	Fauquier	Epidosite	(2)	194	1.32	6.0	6.7	18.3	(2)
15382	Marshall	do	Altered granite	(2)	162	2.16	6.6	6.1	18.0	8
14583	(1)	Frederick	Limestone	(2)	171	.10	8.4	4.8	15.3	3
14879	Winchester	do	Dolomite	(2)	173	.28	3.5	11.4	16.7	14
14880	do	do	Calcareous slate	(2)	158	3.16	8.9	4.5	0.0	5
14969	do	do	Dolomite	(2)	176	.21	3.0	13.3	17.3	15
14463	Pembroke	Giles	Siliceous dolomite	(2)	177	.30	3.2	12.5	16.3	9
14493	Ripplemead	do	do	(2)	175	.52	2.7	14.8	17.3	13
13917	Stickleyville	Lee	Limestone	(2)	168	.34	6.1	6.5	16.3	4
13918	do	do	Argillaceous limestone	(2)	169	.30	6.0	6.6	15.7	5
14584	(1)	do	Siliceous limestone	(2)	168	.30	5.7	7.0	14.7	7
14197	Point of Rocks	Loudoun	Sericite gneiss	(2)	168	.36	5.7	7.0	19.3	12
14198	do	do	Limestone conglomerate	(2)	175	.33	4.5	8.9	(2)	(2)
14284	Between Blacksburg and Newport.	Montgomery	Feldspathic sandstone	(2)	161	1.73	(2)	(2)	16.7	8
15152	Arrington	Nelson	Amphibolite	(2)	185	1.42	3.5	11.4	18.3	14
14384	(1)	Pittsylvania	Biotite gneiss	(2)	170	.41	4.5	8.9	18.0	6
15366	Occoquan	Prince William	Altered granite	(2)	163	1.06	4.7	8.5	19.3	8
14461	(1)	Rappahannock	do	(2)	163	.70	3.8	10.5	18.3	12
14988	(1)	Roanoke	Dolomite	(2)	176	.64	5.1	7.8	16.0	7
14017	Timber Ridge	Rockbridge	Siliceous dolomite	(2)	182	.31	2.8	14.3	17.9	12
15112	do	do	Siliceous limestone	(2)	177	.18	3.8	10.5	18.3	21
15113	do	do	do	(2)	175	.13	3.0	13.3	17.7	14
		WASHINGTON.								
15398	(1)	Whatcom	Altered trachyte	(2)	(2)	(2)	(2)	(2)	(2)	15
		WEST VIRGINIA.								
13886	Belington	Barbour	Sandstone	(2)	147	2.60	7.7	5.2	17.3	7
14263	Martinsburg	Berkeley	Limestone	(2)	169	.15	(2)	(2)	14.7	6
14264	do	do	Diabase	(2)	187	.24	(2)	(2)	18.7	20
14265	do	do	Limestone	(2)	170	.08	9.0	4.4	14.7	3
14266	do	do	do	(2)	169	.09	7.3	5.5	14.0	3
14358	Kingston	Fayette	Feldspathic sandstone	(2)	156	3.06	9.3	4.3	14.3	9
13908	Hamlin	Lincoln	Sandstone	(2)	153	4.98	8.7	4.6	12.0	5
13992	do	do	do	(2)	156	3.90	8.6	4.6	3.3	3
14625	Logan	Logan	Calcareous sandstone	(2)	169	.86	7.5	5.3	13.3	7
14733	Berkeley Springs	Morgan	Quartzite	(2)	164	.41	2.6	15.4	19.3	22
14710	Wheeling	Ohio	Blast furnace slag	(2)	130	6.96	7.0	5.7	(2)	(2)
14320	Hurricane	Putnam	Feldspathic sandstone	(2)	153	4.47	11.7	3.4	2.7	4
14133	Beckley	Raleigh	do	(2)	159	2.62	3.8	10.5	15.3	7
14134	do	do	Sandstone	(2)	153	3.03	4.5	8.9	17.2	6
14176	Sylvia	do	do	(2)	160	1.69	6.0	6.6	14.8	6
14245	Spencer	Roane	Calcareous sandstone	(2)	165	1.32	4.7	8.5	15.0	9
13887	Middlebourne	Tyler	Sandstone	(2)	154	4.15	7.8	5.1	9.3	5
14281	East of Littleton	Wetzel	Feldspathic sandstone	(2)	144	6.22	19.1	2.1	9.0	3
14282	do	do	do	(2)	151	4.64	10.6	3.8	6.7	4
		WISCONSIN.								
14132	Brillion	Calumet	Dolomite	(2)	173	.98	4.6	8.7	12.5	4
14759	Madison	Dane	do	(2)	(2)	(2)	(2)	(2)	16.0	(2)
14676	Montello	Marquette	Granite	(2)	164	.25	2.4	16.7	18.7	16
		CANADA.								
13920	Burnt River	Ontario Province	Siliceous limestone	(2)	174	.68	3.1	12.9	15.7	11

[1] Exact locality not known.

[2] Test not made.

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Public Roads, Vol. I, No. 11. Tests of Road-Building Rock in 1918.

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