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SUPERELEVATION AND EASEMENT AS APPLIED TO HIGHWAY CURVES

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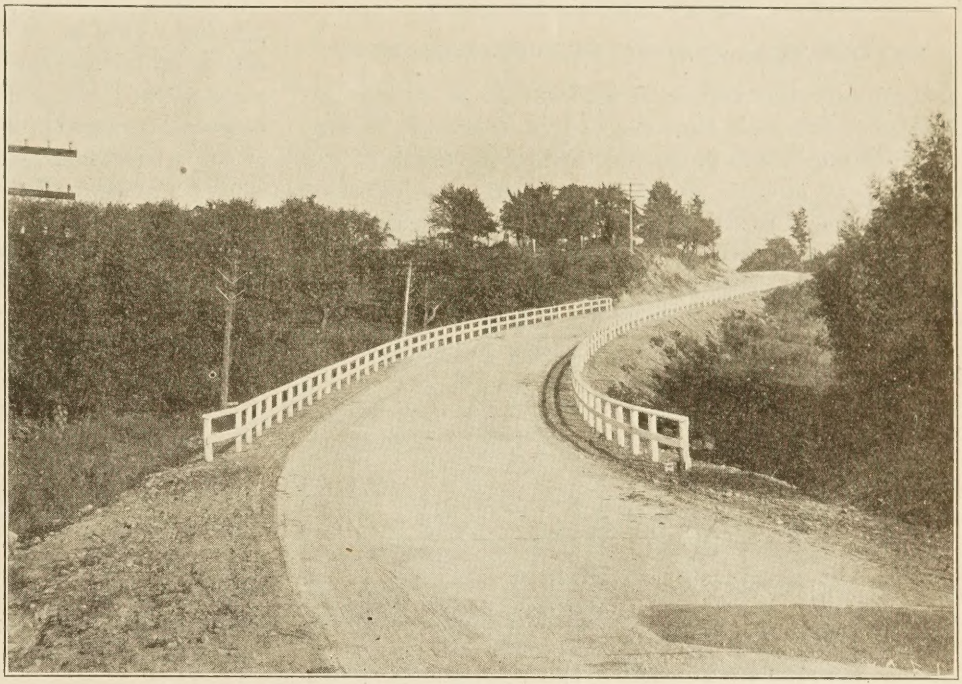


ON WIDENED CURVES WHICH ARE IMPROPERLY BANKED THE TRAFFIC SWINGS TO THE INSIDE OF THE CURVE TO TAKE ADVANTAGE OF THE CROWN, LEAVING AN UNTRAVELED LUNE ON THE OUTER EDGE.

UNTIL recently, highway builders have given very little attention to the superelevation and widening of curves. As long as all vehicles were horse drawn there was, of course, little need for either and, for some time after the automobile brought about a general speeding up of highway traffic, it was thought by many engineers that the omission of these features would do more to promote safety by discouraging speed on the curves than would their inclusion by making higher speeds safe. No doubt the fact that many curves on the older highways were extremely sharp, as judged by present standards, and the line of sight often obstructed by embankments or vegetation, has made the discouragement of speed desirable. However, that, in practice, this theory has not been productive of safety is demonstrated by the alarming multiplication of accidents where it has been applied. Its failure seems to be due to the fact that automobiles are not driven much slower on the curves than they are on tangents, and that there is a marked tendency for drivers who belong on the outside to seek the inside of the curve for, by swinging to the inside, the line over which a vehicle is driven is considerably flattened. Moreover, when this is done, the crown produces the same effect as a certain amount of superelevation, whereas, when a car is driven on the outside of the curve, the crown acts with the centrifugal force and tends to cause skidding. On blind curves this practice results in collisions. These may be largely avoided by recognizing this tendency and providing curves of ample radius and proper superelevation so that it is as safe and as comfortable to follow the outer edge of the pavement as to cross to the inner edge. Drivers can then, without inconvenience, keep to their own side of the road or if, carelessly, they take the wrong side, they can swing back to the proper side in emergency, without danger of accident.

The railroad builder has long provided superelevation and easement on all curves. His reason for so doing has lain partly in the fact that easement and superelevation reduce the cost of maintaining curves, partly in the fact that easement curves favorably effect the wear and tear on equipment, but principally in the fact that easement and superelevation are necessary for the comfort of the passengers traveling on the fast trains. For this latter reason the tendency on the part of most railroads has been to base both easement and superelevation on the speed of the express trains, thereby insuring the greatest comfort for the higher classes of traffic. For the highway builder the problem is a little more complex. Highway traffic is made up of a variety of vehicles capable of and habitually propelled at widely varying speeds. It must also be recognized that these vehicles are propelled in different ways and carried on tires having widely different characteristics. Moreover, the smoothness of the surface of the various types of highway pavements leads to differences in the tendency of these vehicles to skid or to slide in rounding curves. It is necessary to consider all of these factors in arriving at a proper amount of superelevation. Stated in different terms, the problem of the highway engineer is not merely one of providing such superelevation that all vehicles proceeding at a legal speed can round these curves without danger, but of so designing curves that horse-drawn traffic, for instance, which moves at only 3 or 4 miles an hour can use the highway without danger of sliding to the inside edge of the pavement. Indeed it is because these two types of traffic establish such opposing limitations on design that an analysis of the problem of superelevation is necessary.

MATHEMATICAL ANALYSIS OF SUPERELEVATION.

Complete superelevation consists in so tilting the surface of the road that the center of gravity of any vehicle using it will be moved toward the inside of the curve by an amount sufficient to balance the effect of the centrifugal force. The amount of the superelevation required in order to balance the centrifugal force depends on the speed at which a vehicle is moving and the radius of the path in which it is traveling. The relationship of these factors is shown in figure 1.

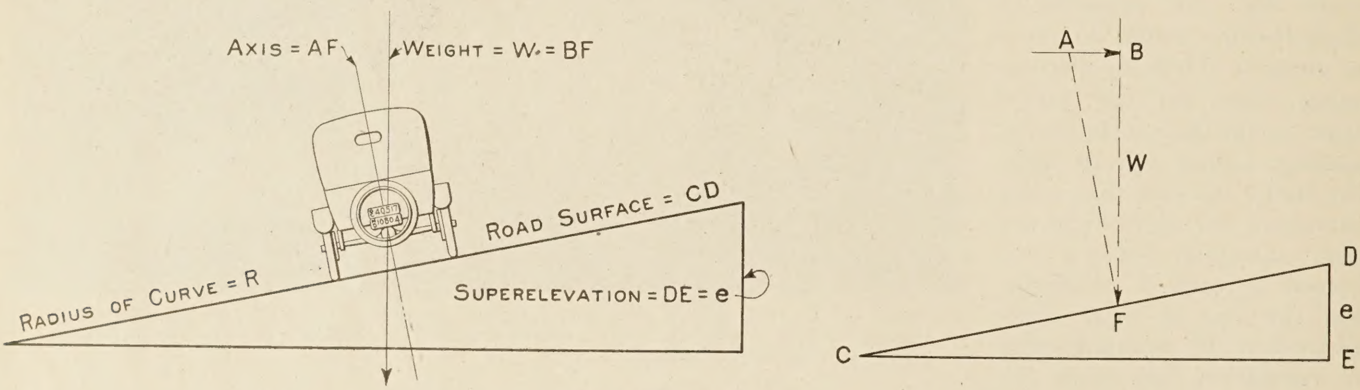


FIG. 1.—Diagram of forces.

According to the mechanics of moving bodies (see fig. 1),

$$AB=\frac{Wv^2}{gR} \tag{1}$$

where AB is the centrifugal force, W the weight of the body in pounds, v the velocity in feet per second, g the acceleration of gravity or 32.16 feet per second, and R the radius of the curve in feet.

From similar triangles in figure 1,

$$AB=\frac{We}{CE} \tag{2}$$

Equating (1) and (2)

$$Wv^2=\frac{gR\cdot We}{CE} \tag{3}$$

or,

$$e=\frac{CE\,v^2}{gR} \tag{4}$$

Substituting 1 foot for CE, reducing v from feet per second to miles per hour, and substituting 32.16 feet for g, there results:

$$e=\frac{V^2}{15R}\text{ (nearly)} \tag{5}$$

in which e is in feet, V in miles per hour, and R in feet. Thus, for a speed of 25 miles per hour (for the relation of other rates of superelevation to various speeds and radii see fig. 2) on a curve having a 200-foot radius, the superelevation theoretically required for the complete counterbalancing of the centrifugal force is 0.21 foot per foot of width, or 3.8 feet for an 18-foot pavement. It is evident that this superelevation exceeds what may be considered a safe amount for the slow moving traffic or, in other words, that such superelevation would cause the slow moving iron-tired traffic to slide toward the inner edge of the road. Hence, it becomes necessary either to establish arbitrary rates of superelevation consistent with the needs of the slower moving traffic or to use curves of longer radius.

Just where such sliding can be expected to take place will depend, of course, on the nature of the surfacing and its condition (that is, whether dry, wet, or covered with frost) as well as on the nature of the tires, but it is safe to say that, as a general practice the pitch of the surfacing should not exceed 10 per cent. Such a pitch, however, will provide complete superelevation for a speed of only 17 miles per hour on a curve having a radius of 200 feet, and almost none of the traffic on our highways moves at a speed as low as this. Indeed a very ordinary speed is from 25 to 30 miles per hour. Moreover, this speed is, generally speaking, a legal speed. Therefore, if the traffic is to be completely protected, even within the legal speed limits, not to mention conditions which will frequently arise as a result of reckless driving, and if, at the same time, a pitch of 1 foot in 10 is the maximum that can properly be adopted in superelevating curves, it necessarily results that where the legal speed is 25 miles per hour the minimum radius of curvature for which full protection can be given is about 415 feet and where a speed of 30 miles per hour is legal the minimum radius of curvature for which full protection can be given is 600 feet.

On the other hand, it is generally assumed that full superelevation is unnecessary. Thus the committee on recommended practices for concrete road and street construction has recommended a maximum superelevation of three-fourths inch per foot of width on curves having a radius of 150 feet or less. This is complete superelevation for a speed of not quite 12 miles an hour on curves having a radius of 150 feet. On curves having a radius of from 150 feet to 500 feet the superelevation recommended is at the rate of one-half inch per foot of width, and for curves having a radius of over 500 feet the normal crown is recommended. These recommendations provide complete superelevation for a speed of 11 miles per hour on a curve of 200-foot radius, for 17 miles per hour on a curve of 500-foot radius, while for curves having a radius of over 500 feet the effect of this recommendation is to trust to the stability of the vehicle itself to resist the total centrifugal force plus the slight addition thereto that results from the fact that the crown on the outer edge of the curve will increase rather than decrease the effect of the centrifugal force and consequently the tendency to skid.

These recommendations have been analyzed in this fashion for the purpose of emphasizing the fact that many authorities, and with entire justice, assume that the adequate protection of the traffic does not imply a superelevation which will counterbalance all of the centrifugal force. Indeed, an analysis of the above recommendations would seem to indicate that the authors of these recommendations have assumed that, if the centrifugal force resulting from a speed of 10 to 15 miles per hour is compensated, the stability of any well-built and properly driven vehicle will take care of all the additional side thrust that results from operating the vehicle within reasonable limits of speed. This is, of course, correct. Moreover, the introduction of superelevation to counterbalance part of the centrifugal force, though correcting the tendency of drivers to hug the inside of the road by rendering the outside as safe as the inside, does not so completely protect the traffic that it tends to correspondingly increase speeding on the curves.

METHODS OF ACCOMPLISHING SUPERELEVATION.

The desired amount of superelevation may be secured in a number of ways, but is generally secured either—

(1) By revolving the surface about the center line as an axis; or

(2) By revolving the surface about the inner edge of the pavement as an axis.

There are some modifications of these two methods but, in most cases, the present practices in superelevation are in all essential points based on one of these methods. These methods are outlined in figure 3.

The effect of these methods upon the grade of the center line and of the ditches is worthy of mention. If the surface is revolved about the center line as an axis the effect will be to lower the inside edge of the pavement and to raise the outside edge. This, in cuts, will depress the grade line of the inner ditch and raise the grade line of the outer ditch. If the surface is revolved about the inner edge as an axis, the effect will be to raise the grade of the center line, causing a slight hump in the grade. However, as the hump falls on the curve and is, therefore, not noticeable, this is a matter of little consequence. This latter method of superelevating curves has the tendency to increase the amount of fill.

RATE OF TRANSITION.

The success of the superelevation of highway curves is considerably affected by the rate of transition that is adopted. No superelevation is required on tangents. As soon as a circular curve is entered the full superelevation is required. Obviously, it is not possible to change from the nonsuperelevated section to the fully superelevated section abruptly at the point where tangent and curve meet. The only course possible is to employ an approach section in which the superelevation will increase from zero to the desired maximum. The proper development of the superelevation in this approach section is almost as important as the superelevation of the curve itself. If the change is made too abruptly the result will be unsightly and the shock of passing from the tangent to the curve disagreeable. Moreover, it is almost as illogical to develop the superelevation on the tangent as it is to develop it after the full circular curve has

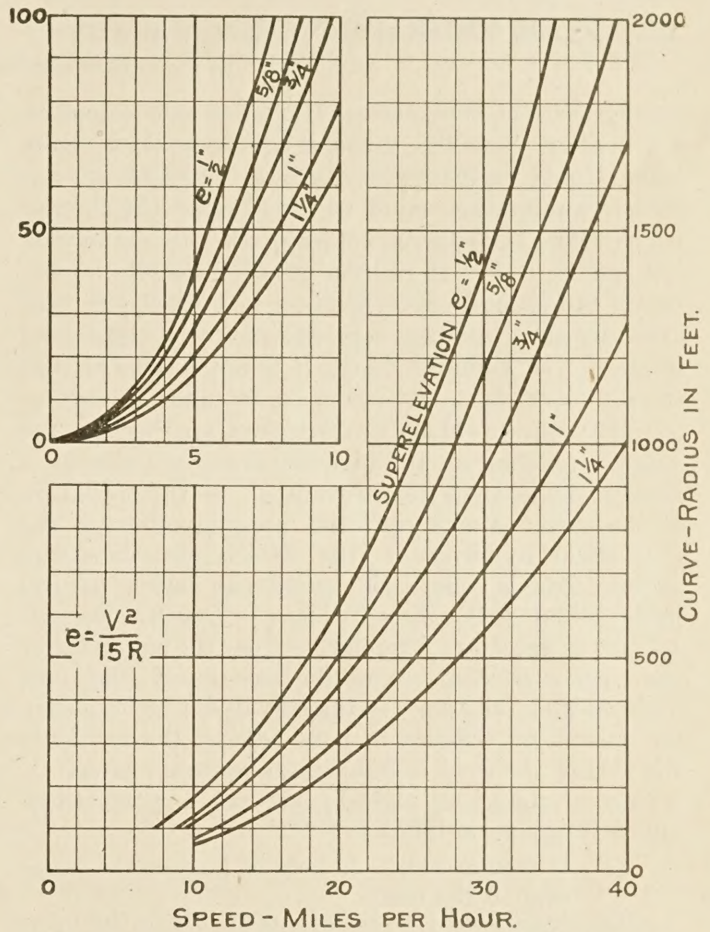
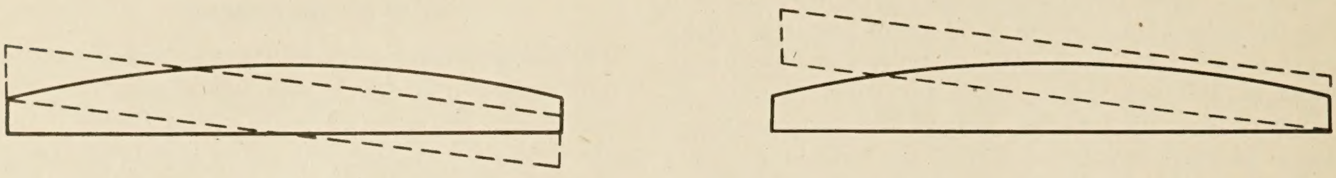


Fig. 2.—Curves showing relation between speed, radius, and superelevation on curves.

been reached. The only correct practice is that which is indicated by the theory as expressed in the equation $e=\frac{V^2}{15R}$. The velocity V being assumed constant, it follows that as the superelevation, e, is increased, the radius, R, should be decreased, which is equivalent to saying that the superelevation should be developed on a transition curve the radius of which should decrease from infinity at the point where the superelevation is zero to a minimum equal to the radis of the circular curve where the superelevation reaches its maximum, i. e., at the P. C. In railroad practice the relation between radius and superelevation is worked out very carefully and conforms very closely to the theoretical relation. In highway design it is not necessary that the radius of the transition curve be made to vary so exactly with the change in superelevation, because the vehicle traveling on the highway is not confined to a fixed track as the railroad train is. If the curvature of the surfaced way conforms approximately to the theoretical requirement, the vehicle, by departing slightly from a concentric course, can take a natural path without being forced to leave the surfaced road. All that is necessary, therefore, is that the curvature of the road shall conform to the theoretical curvature with sufficient accuracy so that it will not be necessary for drivers of vehicles to pass beyond the center or the side of the road in following a natural course.



REVOLVED ON CENTER LINE OF SUBGRADE AS AXIS

REVOLVED ON INNER EDGE OF PAVEMENT AS AXIS

FIG. 3.—Methods of accomplishing superelevation.

Current American highway practice employs three approximations, as follows:

(1) A circular curve of long radius on the inner edge of the road.

(2) Compound circular curves in which the principal circular curve is joined to the tangent by means of one or more circular curves of longer radius.

(3) Spiral transition curves, similar to these commonly used in railroad work.

A satisfactory method, which has, for some reason, not been used in this country, consists in the use of parabolic instead of circular curves. The parabola provides its own transition.

TRANSITION NEEDED ON OUTSIDE.

Whatever form of transition curve is adopted, there is just as much need for the use of it on the outer edge as on the inner edge of the pavement. One of the principal reasons for superelevating curves, as previously pointed out, is to prevent the cars which belong on the outside of the curve from cutting to the inside. To accomplish this purpose, the outside of the curve should be made fully as safe and as easy to drive around as the inside, else the motorist will take the easier course offered by the inside. This means not only that the fully superelevated section of the circular curve should be a plane surface but that the approach to full superelevation should be approximately the same on both sides of the road, and that approximately the same relation of superelevation to radius of curve should be maintained on both edges in the transition section.

Desirable as it is to maintain like conditions on both sides of the road, it is a fact that the present practice does not provide this condition. Without exception the State standards provide no transition curve for the outer edge; and some of the States approximate the transition requirements of the inner edge by providing a circular curve of long radius, a practice which leaves it to the motorist to make his own transition on the widened curve which results. It is, of course, impossible to ease the outer edge of a curve treated in this manner, since the use of a parallel curve on the outer edge would simply result in a road with a circular curve, although it be of somewhat longer radius.

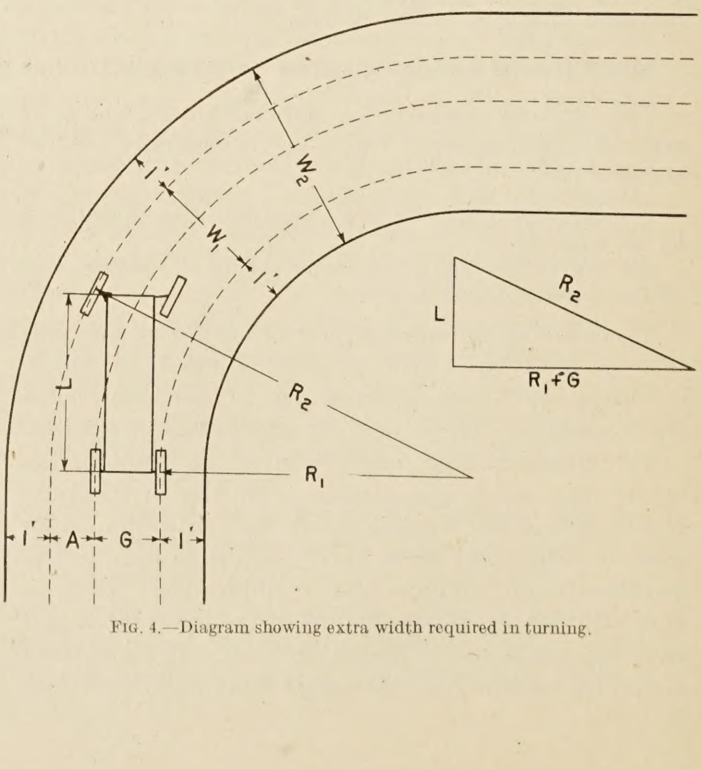


FIG. 4.—Diagram showing extra width required in turning.

Not only does the present practice overlook the need for transition on the outside of curves, but by providing for the elimination of the crown between the P. T. C. and the P. C. it creates an unfavorable condition of superelevation for the car which is traveling on the outer edge in this section in which the superelevation is being developed. Approximately half of the transition section is required to eliminate the crown. In this distance, the motorist, who has begun to round the curve at the point where the transition curve begins on the inner edge, has the advantage of no superelevation if he continues along the outer edge. The result is that he tends to move over to the inside of the curve, and when the full circular curve is reached he is found well toward the center. Fortunately the added width of pavement which results from the use of the transition curve on the inside and the plain circular curve on the outside, leaves room for passing cars, but the extra width on the outside is practically useless.

This matter sums itself up in the simple statement that both transition and superelevation should apply to the inside and the outside of the curve alike.

FLAT SECTION AT P. T. C. ADVISABLE.

But to bring this about it is necessary to start at the P. T. C. with a crownless section and to carry this section to the P. T. T. Moreover, the simplest plans are the most accurately carried out in the field and therefore it is well to have the crown reduced to zero at the P. T. C. and built up from zero at the P. T. T. from a purely construction standpoint. This can be rather simply accomplished by raising the subgrade at the P. T. C. by an amount equal to the difference between the crown thickness and the thickness of the flat slab section and setting the forms to taper back. By allowing 10 feet per inch of crown height, skilled workmen can change from the full crown to a crownless section in a very acceptable manner, performing the work entirely by eye. This method applies to macadam and gravel roads as well as to roads of the hard-surface types.

The amount of crown that is used on the various types has been more or less fixed by custom but is, in theory at least, governed by practical considerations involving the drainage of the surface of the road. There has, therefore, been some objection to the creation of even short sections of highway not liberally crowned. This objection can be carried too far. Indeed, in designing curves, it has often resulted in plans which have been so difficult of accomplishment under normal field conditions that they justify the impression that, in spite of the common objections to a crownless section the greater accuracy of construction which will result from arriving at the P. T. C. with a flat section will more than justify the general adoption of this procedure. There can, of course, be no objection to this as applying to hard surfaced roads, for in such types the immediate drainage of the road surface is of minor importance.

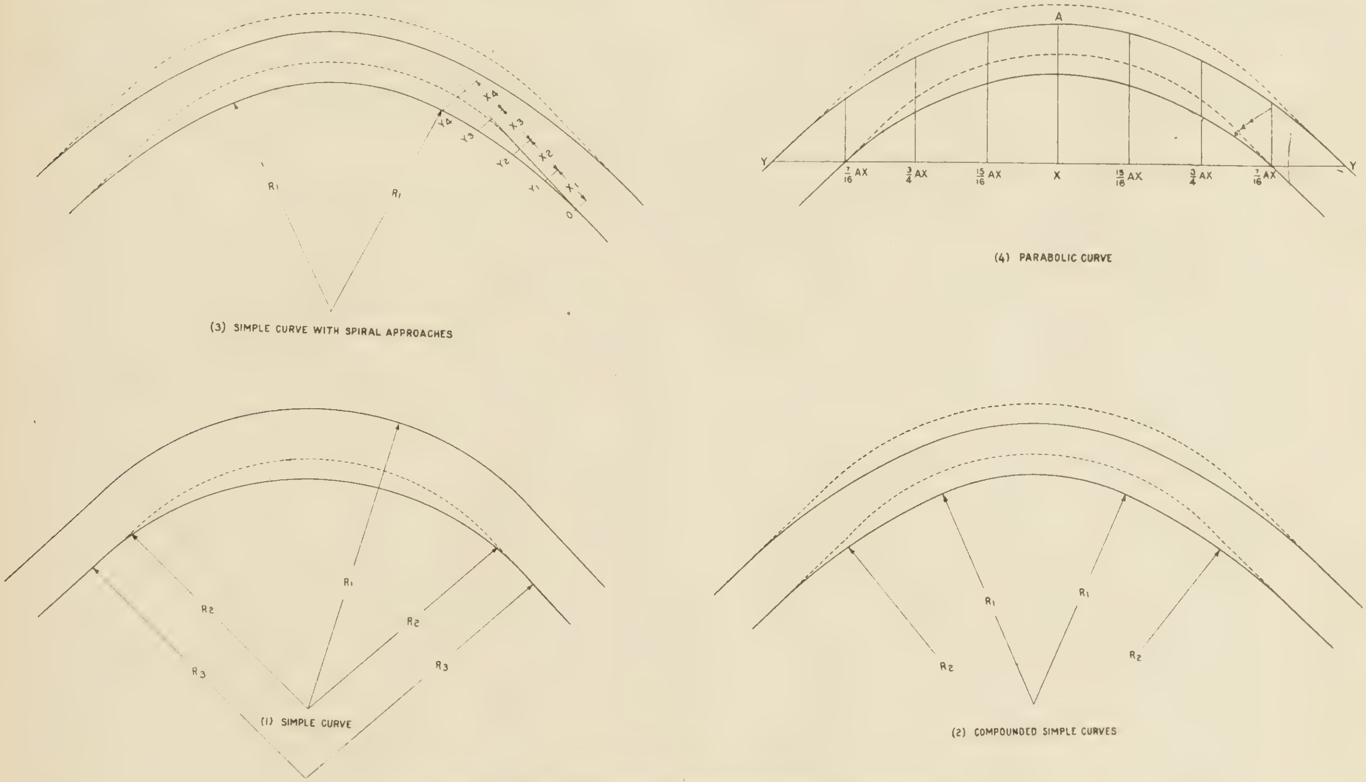


Fig. 5.—Types of transition curves and method of widening.

COMPLETE SUPERELEVATION NOT NEEDED.

It has previously been shown that the equation of superelevation is $e = \frac{V^2}{15R}$ from which it naturally follows that for any fixed value of V, that is, for any given speed, e should vary inversely as the factor $15R$. Curves must, of course, be designed for one speed, and where there is a legal speed limit, they should be designed so that that speed is actually a safe speed. As previously pointed out, this does not imply that the superelevation shall fully counterbalance the centrifugal force at that speed, but it does imply that there shall be a policy as to the relation of these factors, and an established minimum of protection offered on all curves. Just what this minimum should be will depend somewhat on local conditions. The present practice of New York State, which provides as a standard a 300-foot minimum radius of curvature and superelevation to compensate for a speed of 20 miles per hour on curves of this radius, with an increasing amount of compensation (see Table 1) for curves of longer radius (i. e., where the unobstructed sight distance is greater) is the highest standard so far established. However, where this high standard can not be adopted, a minimum radius of curvature of 200 feet with superelevation to compensate for a speed of 15 miles an hour, and increasing compensation for curves with longer unobstructed sight distances should prove quite satisfactory.

Let it be assumed that it has been established that no curve will be constructed the superelevation on which will compensate less than 15 miles per hour. If, then, a curve of a radius of 200 feet is to be built and the transition is to be composed of two curves having radii of 800 feet and 400 feet, the superelevation should be 0.019 foot per foot of width for the first curve, 0.038 foot per foot of width for the second curve, and 0.075 foot per foot of width for the full curve. For all practical purposes a linear increase in the superelevation from 0 at the P. T. C. to 0.075 foot at the point of full curvature would, in this case, be satisfactory and, by many engineers, preferred to any other system. Such a transition should apply, with proper adjustment for the longer radii to the outside as well as the inside of the pavement.

If a closer control is desired it can be readily obtained by using either spiral transition curves or parabolic curves, instead of compound circular curves. The peculiar virtue of these curves lies in the fact that they closely approach the path actually traversed by a fast-moving car. It takes an appreciable time to operate the steering device on an automobile. During this time the car is moving rapidly ahead. The result is not a circular path, but one the radius of which constantly changes as the wheel is turned. From the standpoint of the driver of such a car the minimum radius to which he must drive is, therefore, of vastly less importance than that he be given a proper space in which to conform the direction in which his car is moving to that radius. Moreover, from the standpoint of the driver, it is just as simple a matter to operate his vehicle on a curve of constantly changing radius as on a curve of uniform radius providing the rate at which the radius changes is within the normal control limits of his car. He would, therefore, prefer to drive over a sharp curve having proper approaches rather than to operate on a curve of longer radius to which there are no approaches. For this reason both the spiral transition and the parabolic curve are pleasant to drive over and, relatively, much safer than simple circular curves. Moreover, as the actual radii of these curves shortens from infinity to the minimum used by well-known mathematical rules the superelevation can be readily adjusted. In the case of the parabola it can be handled, where superelevation is by revolution on the inner edge, by the simple process of treating the elevation of the outer edge as a parabolic vertical curve having its apex at the apex of the horizontal curve.

Short parabolic curves are the simplest of all curves to lay out. They depart from standard American practice, however, in that they are chained in—no instrument work being required—and that, in practice, they are generally figured in the field—no tables being used. This has, no doubt, prejudiced some against their use but as both the calculations and the chaining are very simple this prejudice rapidly disappears wherever they are introduced. Spiral transition is so fully treated in texts on railroad engineering that it will not be discussed here except to say that, for curves as sharp as those used in highway work, the spiral transition sections will usually be laid off by offsetting from the tangent. Where spiral transition curves are used, as where compound circular curves and parabolic curves are used, the outside edge of the pavement as well as the inside edge should be treated.

No matter what kind of curves are used, the length of the transition should vary inversely with the radius of curvature of the full curve. There are, at present, no well defined standards governing this matter but current practice suggests that it will probably be found that a 100-foot transition section will give easy transition to a curve of 200-foot radius. This is equivalent to a period of nearly 3 seconds in which to divert a vehicle moving at the rate of 25 miles an hour, from a straight course to a circular course of the above radius. For curves of longer radius, the diversion is not as great and the transition can be correspondingly decreased.

WIDENING OF CURVES.

Because of the fact that the rear wheels of ordinary vehicles are fixed causing them, when rounding curves, to travel on a different radius from the front wheels it is advisable that some added width of surface be provided on curves of short radius. This

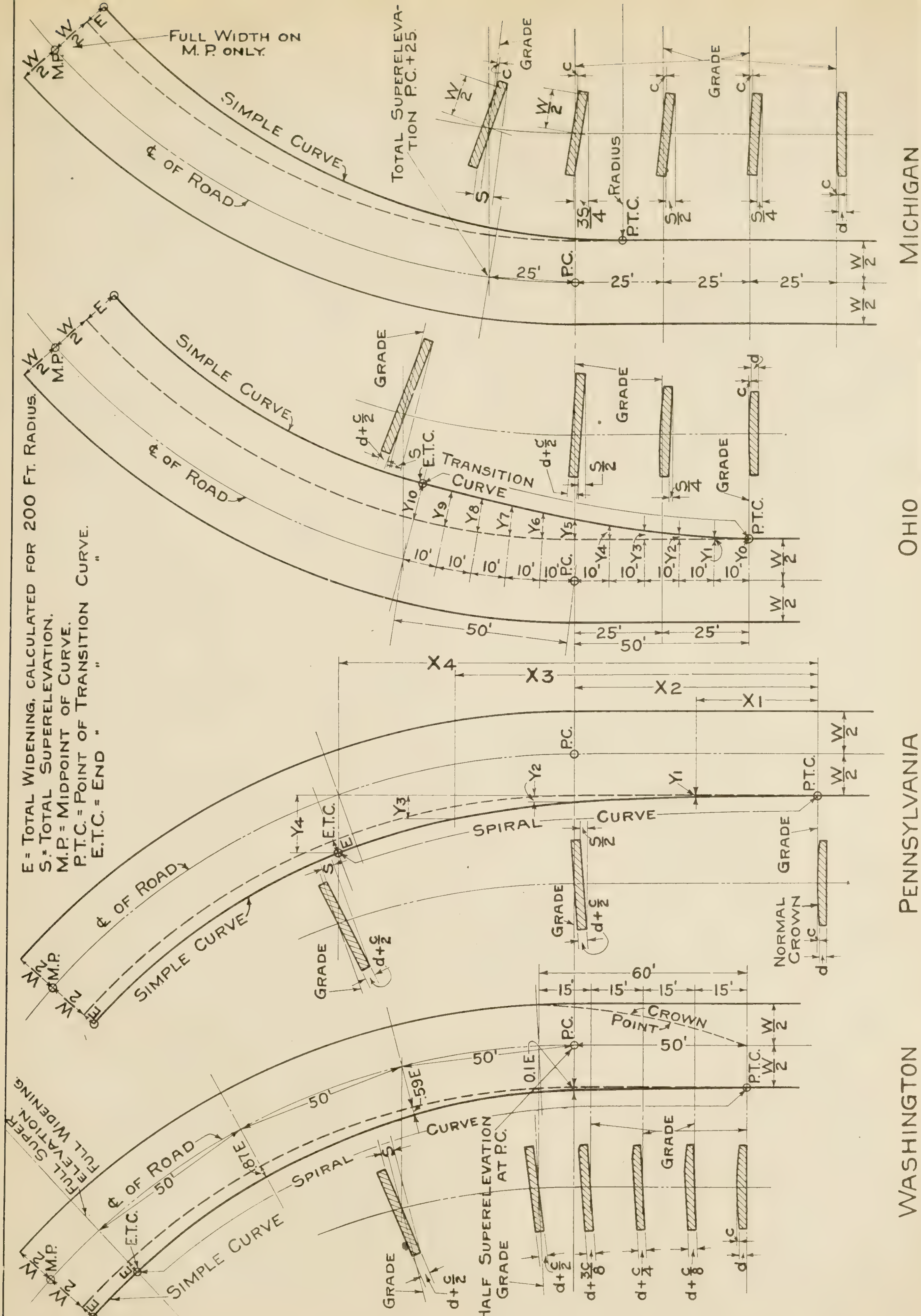


Fig. 6.—Typical methods of superelevation and widening in use by the States.

added width should vary with the radius of the curve, the gauge of the wheels and the length of the vehicle as outlined in figure 4. By consulting this figure it will be seen that,

$$R_2^2=(R_1+G)^2+L^2 \qquad W_1=R_2-R_1 \qquad A=R_2-(R_1+G)$$

in which

R_2 = Radius of inner rear wheel curved track in feet.
R_1 = Radius of outer front wheel curved track in feet.
G = Gauge of wheels in feet.
L = Wheel-base length in feet.
W_1 = Full width required for clearance in feet.
A = Amount of widening required in feet.

Thus, a touring car having a 136-inch wheel base and 56-inch gauge, turning on an outer front wheel radius of 21.2 feet, would be turning on an inner rear-wheel radius of 13.2 feet and would require an added width of 3.3 feet. Similarly, a 5-ton motor truck of 168-inch wheel base, turning on an inner rear-wheel radius of 25 feet would require an added width of 3.1 feet to maintain the same clearance provided for on tangents. Fortunately, the radii of the curves usually used are greater than the 13.2 feet and 25 feet assumed in these examples, but they illustrate why it may be advisable to widen the pavement on sharp curves. For a 150-foot radius, the added widths would be 0.3 foot for the touring car, and 0.4 foot for the 5-ton truck.

The ordinary procedure in widening the pavement on curves is to increase the width by an amount which is considerably greater than what is theoretically required to compensate for the fact that the wheels of a vehicle do not track. Where such widened curves are properly banked it will be observed (1) that the added width on the inside of the curve is being constantly used (2) that both the inside and outside lines of traffic are proceeding in paths that resemble a parabola, and (3) that a width on the outside of the pavement corresponding approximately to the addition on the inside of the pavement is little used. The natural deduction from these easily checked statements is that the widening of the pavement on ordinary curves is of relatively little importance, but that it is a matter of real importance that curves be so approached that vehicles shall have a reasonable distance within which to adjust their course to the curvature in the alignment of the road.

As a matter of fact, the reason that vehicles do not use the outside of widened curves is that, generally, there is no transition between the tangent and the full circular curve on the outer edge of curves as they are now designed. As a result the outer line of vehicles, in making its own transition, cuts in from the outer edge of the pavement, leaving the extra width practically unused. The conclusion from this is that the only widening that is required is that which is necessary to provide turning space for the rear wheels plus a small addition to satisfy the psychological demand for more passing room on curves. If, with this small additional width, carefully designed approaches were provided on the outside as well as the inside of the curves, the result would be perfectly satisfactory and there would be a distinct saving in the amount of pavement laid on each curve, the saving being in the general shape of a lune extending from the P. T. C. to the P. T. T. and having, as its maximum width, a large part of the extra width now laid on the inside of curves.

CONSTRUCTION METHODS.

The execution, in the field, of superelevation and widening requires care and skill. Indeed, an engineer should always be employed to stake out work of this character and to keep check on it during construction. Where curves are widened the widening should be made a part of the pavement and built simultaneously with the rest of the pavement.

The greatest difficulty in constructing superelevated curves occurs in the section where the crown is eliminated, and unless these sections are carefully built they are apt to be rough and the comfort of the traveler as well as the permanance of the road adversely affected. Where the crown is eliminated before the transition section is reached the elimination is generally done by eye, and both the transition and the superelevation are tended to by an adjustment of the side forms.

This method is applicable to all types of pavement. Where it is used for gravel or macadam pavements stakes instead of side forms are set both on the inside and the outside of the pavement and the subgrade prepared and the metal spread in accordance with these stakes. Of course, no very great accuracy can be secured with pavements of this type, as surfaces compacted by rolling are always somewhat wavey, but with a moderate amount of experience and with care in calculating the amount of material which must be spread in order to secure a given thickness of finished road, the results will be entirely satisfactory.

In taking out the crown, when this is done before the superelevation is begun, an allowance of 10 feet per inch of crown is ample. Where the elimination of the crown and the transition are combined this is sometimes accomplished on concrete roads by the use of false forms, and efforts have been made to use adjustable templets. The adjustable templet is ideal in theory, but in practice it has been found difficult to manipulate and has been generally abandoned. Beyond the section in which the crown is eliminated there is no difficulty. The practice of using no crown where there is full superelevation is universal and the added width offers no problem of importance. The forms can be, and indeed are, set

State practices in superelevation and easement of curves.

State.	Method of superelevating.	Pitch of fully superelevated sections.	Transition section and rate.	Complete bank.	Pavement cross section on fully superelevated section.	Extra width.	
						Amount.	Character.
California mountain roads.	....	3/4 inch per foot, 75-foot radius and under. 1/2 inch per foot, 75-100-foot radius. 3/8 inch per foot, 100-150-foot radius. 1/4 inch per foot, 150-225-foot radius. 1/8 inch per foot, 225-300-foot radius.	Transition length 30 feet, one-half on tangent and one-half on curve, except on reverse curves, where it may be made as long as 50 feet.	....	Uniform slope, thickness and width, except that when superelevation exceeds 1/4 inch per foot and is toward the fill slope a curb 6 inches wide and 10 inches high is used on the lower edge.	2 feet on 150-foot radius and less, if pavement is less than 18 feet wide.	
Indiana	Revolved on center line of surface as axis.[1]	1/2 inch per foot width, all curves.	Outer edge raised from 2 inches below elevation of center line at P. T. C. to 4 1/2 inches above center line at P. C. and line on inner edge, 9 feet from center line, lowered from 2 inches below elevation of center line at P. T. C. to 4 1/2 inches below at P. C.	Full bank P. C. to P. T.	Uniform slope edge to edge, and uniform thickness, P. C. to P. T. Uniform width. P. C.+50 to P. T. −50.	8 feet for 30° curve .... 8 feet for 20° curve .... 6 feet for 15° curve .... 4 feet for 10° curve .... 2 feet for 5° curve .... 1.6 feet for 4° curve	Uniform width P. C. +50 to P. T. −50. Transition curves, 100 feet long.
Michigan	Revolved on a line near inner edge of curve parallel to center line of unwidened pavement and one-half of the normal width of the pavement therefrom.	1 inch per foot width, 66 to 150 foot radius. 3/4 inch per foot width, 151 to 300 foot radius. 5/8 inch per foot width, 301 to 450 foot radius. 1/2 inch per foot width, 451 to 700 foot radius.	P. C.−75, normal section .... P. C.−50, 1/4 normal superelevation. P. C.−25, 1/2 normal superelevation. P. C., 3/4 normal superelevation. P. C.−25, full superelevation.	Full bank P. C.+ 25 feet to P. T.−25 feet.	Uniform slope edge to edge, and uniform thickness; variable width.	6 feet for 66 to 150 foot radius. 5 feet for 151 to 300 foot radius. 4 feet for 301 to 450 foot radius. 3 feet for 451 to 700 foot radius.	Transition and widening obtained by using simple curve of longer radius on inside of curves.
New Jersey	Revolved on center line of surface as axis.	1/3 inch per foot width	100-foot easement section .... Normal section at P. C.−100, one-half of full superelevation at P. C.−50, and crown largely eliminated.	Full bank P. C. to P. T.	Uniform slope edge to edge, uniform thickness, and uniform width.	....	No widening.
New York	Revolved on center line of surface axis.[1]	1 inch per foot width up to 300-foot radius. 15/16 inch per foot width, 300 to 400 foot radius. 7/8 inch per foot width, 400 to 550 foot radius. 13/16 inch per foot width, 550 to 800 foot radius. 3/4 inch per foot width, 800 to 1,500 foot radius. 1/2 inch per foot width, 1,500 to 2,500 foot radius.	Zero at 85 feet back of P. C. (300-foot radius). Zero at 75 feet back of P. C. (300 to 400 foot radius). Zero at 65 feet back of P. C. (400 to 550 foot radius). Zero at 55 feet back of P. C. (550 to 800 foot radius). Zero at 45 feet back of P. C. (800 to 1,500 foot radius). Zero at 35 feet back of P. C. (1,500 to 2,500 foot radius). Transition on tangents	Full bank P. C. to P. T.	....	8 feet for 300-foot radius...... 7 feet for 300 to 400 foot radius. 6 feet for 400 to 550 foot radius. 5 feet for 550 to 800 foot radius. 4 feet for 800 to 1,500 foot radius. 3 feet for 1,500 to 2,500 foot radius.	Uniform width P. C. to P. T. with transition on tangents.
Ohio	Inside edge of pavement kept a uniform distance, equal to crown on tangents, below grade line.	0.042 foot per foot width, 30° curve. 0.042 foot per foot width, 4° curve. 0.032 foot per foot width, 3° curve. 0.021 foot per foot width, 2° curve. 0.011 foot per foot width, 1° curve.	At P. C.−50 feet, normal crown. At P. C.−25 feet, pavement raised one-fourth total superelevation. At P. C., pavement raised one-half total superelevation. At P. C.+50 feet, use total superelevation.	Full bank P. C.+ 50 feet to P. T.−50 feet.	Uniform slope edge to edge, and uniform thickness; uniform width, P. C.+50 to P. T.−50.	8 feet for 30° curve .... 1.6 feet for 4° curve, etc. ...... 0.4 foot for 1° curve	Uniform width P. C.+50 feet to P. T.−50 feet; transition curves 100 feet long, established by offsets.
Pennsylvania ...	Revolved on center line of surface as axis.[1]	1/2 inch per foot width for all curves, 50 to 300 feet radius.	Spiral transition used on inside only. Rate and amount by special table.	Full bank between transition curves.	Uniform slope edge to edge, and uniform thickness, uniform width.	6.5 feet for 50-foot radius .... 4.5 feet for 100-foot radius.... 3 feet for 200-foot radius...... 2.5 feet for 250-foot radius.... 2 feet for 300-foot radius......	Spiral transition; uniform extra width between transition curves.
Washington	Revolved on center line of subgrade as axis.[1]	0.055 foot per foot width, 40° curve. 0.0475 foot per foot width, 20° curve. 0.035 foot per foot width, 10° curve. 0.018 foot per foot width, 3° curve.	At P. C.−50 feet normal section; at P. C. one-half of total superelevation; at P. C.+50, full superelevation transition curve on inside edge of pavement only.	Full bank (P. C.+50 feet to P. T.−50 feet).	Uniform slope and thickness; width increases to P. C.+150 feet, then uniform to P. T.−150 feet.	Extra width equals the cube root of the degree of curvature to the nearest one-tenth foot, the minimum used being 1.5 feet.	Zero at P. C.−50 feet. 10 per cent of total at P. C. 59 per cent of total at P. C.+50 feet. 87 per cent of total at P. C.+100 feet. Total at P. C.+150 feet.
West Virginia ..	Revolved on center line of surface as axis.[1]	0.06 foot per foot width, 75-foot radius. 0.047 foot per foot width, 125-foot radius. 0.039 foot per foot width, 175-foot radius. 0.037 foot per foot width, 200-foot radius. 0.025 foot per foot width, 400-foot radius.	Superelevation at P. C. one-third total, and at P. C.+25 two-thirds total.	....	Uniform slope edge to edge, and uniform thickness.	3 feet for 75-foot radius...... 2 feet for 125-foot radius..... 1 foot for 175-foot radius..... 0 foot for 200-foot radius..... 0 foot for 400-foot radius.....	Widening to be done by "mooning" on inside of curve.

[1] "Center line" referred to in Indiana, Pennsylvania, New York, Washington, and West Virginia is the center line of the unwidened curve, and not the center line of the curve after widening.

to the established grade and concrete manipulated as on other parts of the work except that, even though a machine finisher is used on other parts of the work, hand methods of finishing have to be used on the curves.

STATE PRACTICES IN SUPERELEVATION.

Although not all of the State highway departments have established definite standards for the superelevation and easement of highway curves, a sufficient number have outlined their practice in their plans and specifications so that a general survey is possible. The table on page 11 gives this information in brief and shows the method of obtaining superelevation, the pitch of the totally superelevated section, the length of transition section and the rate of transition, together with the amount and character of the extra width of pavement used in connection with superelevation for the States listed.

METHOD OF SUPERELEVATION.

As previously pointed out, the superelevation is accomplished by revolving the pavement on the center line as an axis or on the inside edge as an axis. A glance through the table indicates that the first method is the one most commonly adopted. This is as might be expected, for under this practice the center-line grade is not distorted and there is no appreciable modification in the quantity of earthwork which must be handled.

THE PITCH OF FULLY SUPERELEVATED SECTIONS.

The pitch of fully superelevated sections varies considerably. In Ohio the maximum rate of superelevation, which applies to all curves of four degrees or sharper is one-half inch to the foot. In New York a pitch of 1 inch to the foot is used where the radius is 300 feet. In Michigan the same pitch is used for a radius of 150 feet or less. Practically all of the States vary the amount of superelevation as the radius of curvature varies. In this regard, as in the amount of superelevation used, the New York State practice deserves special study. By comparing the New York practice with the rates of superelevations and the radii shown in figure 2, it will be observed that the New York practice furnishes complete superelevation for a speed of 20 miles per hour on curves of 300-foot radius, and a complete superelevation for a speed of 30 miles per hour on curves having a 1,500-foot radius.

Speaking generally, the protection offered on curves whose radii lie between these limits is from 20 to 30 miles per hour. Speed limits in New York State are somewhat higher than in most States, but these facts serve to indicate that the policy in New York is to superelevate highway curves so that the traffic will be almost completely protected within the legal speed limits except on curves which are so short that drivers would normally choose lower speeds.

TRANSITION SECTION AND RATE.

In the matter of transition, as in other matters, State practice varies considerably. The length of the transition section should be such that a car moving at the legal rate of speed, say 25 miles per hour, can be comfortably diverted from the tangent to the full curve within the transition section. As a speed of 25 miles per hour is equivalent to a speed of 36 feet per second, it can be readily seen that transition sections of some length are necessary. In Ohio the transition section is 100 feet long without reference to the radius of the curve to which it is an approach. Other States generally follow a similar system. Indeed, it is only in New York State that the length of the transition section is a function of the radius of curvature. Logically, is takes longer to divert a car from a tangent to a curve of 300-foot radius than to a curve of 2,500-foot radius and this fact has been recognized by the New York authorities to the extent of requiring an 85-foot transition section in the first case as against a 35-foot transition section in the second case.

FULLY SUPERELEVATED SECTIONS.

Without exception the States which superelevate use a uniform slope from edge to edge and a uniform thickness where curves are fully superelevated. The thickness of the pavement varies from project to project in many States, and as traffic conditions change, in all States. Therefore, no definite statement as to thickness can be made. In general, however, the practice is to use a thickness on the superelevated sections equal to the edge thickness on the tangent plus one-half of the crown on tangents.

EXTRA WIDTH OF SURFACE.

There is as much variation in regard to the widening of curves as in regard to any other matter. As stated above, the widening of a curve has the effect of flattening the path used in driving, which is equivalent to lengthening the radius. If the transition is carefully worked out widening can be legitimately omitted except under the circumstances previously discussed in this article as applying principally to curves of very short radius. Thus in West Virginia no additional width is used where the radius is 200 feet or greater and a maximum additional width of 3 feet is used where the radius is 75 feet. This is only the amount necessary to take care of the failure of the hind wheels of an ordinary vehicle to track with the front wheels increased by a reasonable margin of safety and is based on the assumption that where the approach to a curve is properly designed and the curve is liberally banked, this is all that is necessary.

On the other hand, from this very conservative and technically accurate position there is considerable divergence in other States. The maximum diverg-

[Concluded on page 18.]

TNT A SUCCESS IN ROAD WORK

By LUKE E. SMITH, Superintendent of Road Construction, Bureau of Public Roads.

After using 200,000 pounds of War Department TNT in road construction, the writer considers it the best all-around explosive for road work, and believes that when it is thoroughly understood it will produce the best results in any kind of work where blasting is necessary.

It is the safest of explosives to handle. Though it is advisable not to handle it without gloves, it has been observed that the majority of powder men soon discard the gloves, and no bad effects are experienced. This practice, however, is discouraged in the use of any blasting powder.

Though water decreases its efficiency, it does not seriously affect it unless it is exposed until it becomes thoroughly saturated. Good results can be secured in wet ground if holes are not left loaded too long.

Large quantities of it have been used in the National Forest road work of the Bureau of Public Roads and its value has been tested in a number of classes of work. We have learned that good results can be obtained in practically all sorts of work, but to obtain the best results the methods of use must be adapted to the peculiar characteristics of the explosive.

METHODS USED IN GRUBBING.

For use in blasting stumps up to 1 foot in diameter it has been found that a single charge under the center or against the tap root is sufficient. The size of the charge, however, must be varied according to the nature of the ground, and as this varies according to locality and even in different sections of the same road, no hard and fast rule governing the amount of the charge can be laid down. In blasting large stumps it is generally necessary to put charges under the main roots as well as under the center. The important thing to remember is that all charges should be well up against the stump and well covered and tamped. Under such conditions it is most convenient to explode the charge with a battery. But if a battery is not available, the following method can be used. Prepare each place for the major charges under the stump; then from these prepare a small connecting trench. Next load the holes and connect them with runs of TNT about as large as a seven-eighths-inch stick of dynamite laid in the connecting trenches. Cover all with fine dirt and make sure that no air gets to the charge in any place. A No. 8 cap and fuse in any one of the major charges will produce complete detonation. The only objection to this method of loading is the possibility of not getting the entire charge tamped so that it is air-tight. It has been found that an excellent way to confine TNT in stump shooting is to use tin cans as containers. They are much better than any kind of tamping bag for pot-hole shooting.

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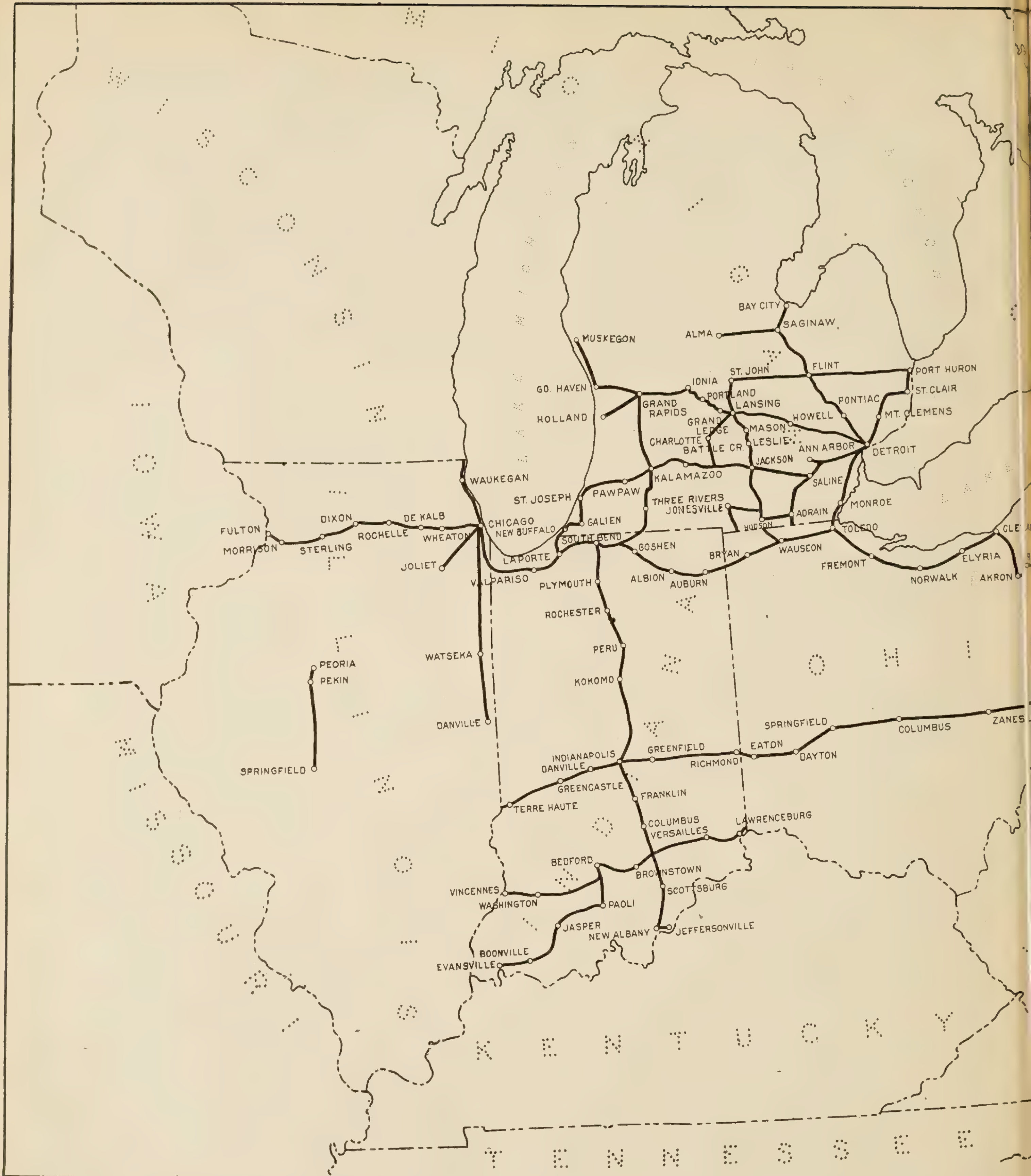
CHEAP DITCHES AND DRAINS.

A great deal of labor can be saved and costs greatly reduced on nearly all classes of work by using TNT to form ditches and drains. The single precaution to be observed to assure success is to keep the charges dry. By using tin cans as containers for the TNT and sinking them at uniform distances the charge will be kept dry even when the ditch runs through wet ground, and a complete ditch will be formed with a minimum of labor. In this case, as in stump blasting, the size of the charge should, of course, be altered with the character of the ground and the size of the ditch desired.

THE BEST EXPLOSIVE FOR ROCK WORK.

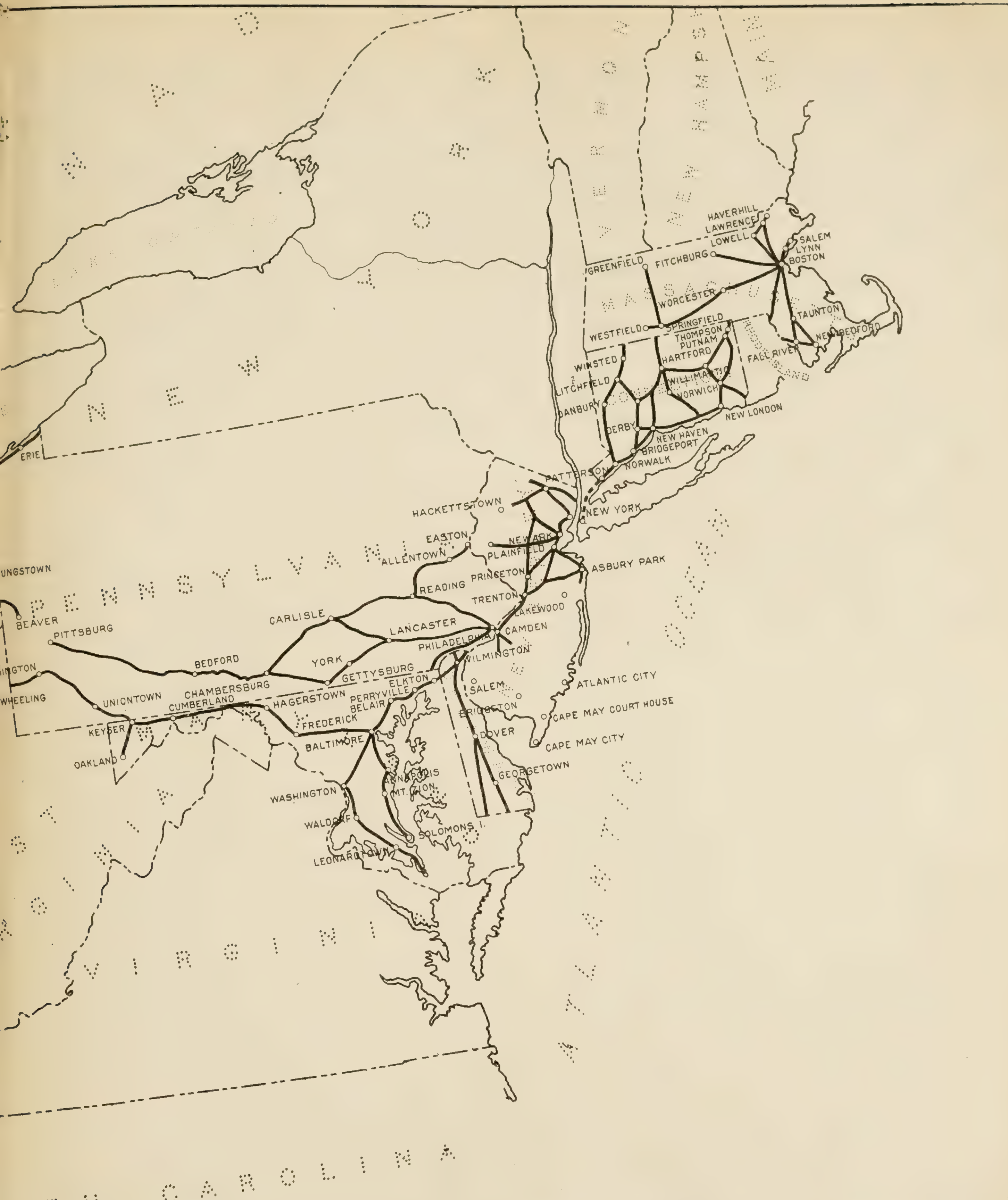
The results of TNT in rock blasting are so far superior to those of any other explosive that we have found that an experienced powderman who has once used TNT can hardly be induced to use anything else. Used in drill holes the results are better beyond comparison than can be obtained with 40 per cent dynamite, regardless of the nature of the rock. All that is necessary is to vary the size of the charge according to the hardness of the rock and the amount of excavation per hole. In extra hard rock it is the best explosive that can be used to spring holes. In springing soft material it has been found advisable to insert a section of gas pipe into the hole long enough to extend to within 1 foot of the bottom, then load the spring charge and fire. The gas pipe keeps the hole from crumbling and the TNT is quick enough to compress the walls of the chambers, which seldom ravel. The success of this practice, however, depends on the skill of the powderman. For crevice blasting TNT has no equal. It exerts a greater moving force than black powder. It seems to have greater shattering power, especially in a downward direction, than other explosives, because its quickness causes the rock to break across the seams without following them.

In some of the work of the bureau the use of TNT has made it possible to dispense with drilling machinery. It has been found that a small charge placed in the slightest depression in the hardest kind of rock will open a pocket or crevice into which a good charge can be placed which will produce the best results. In moving rock by this method during the past season 2 pounds of TNT per cubic yard were used on one project. At 50 cents per pound the cost of the explosive for this work would be $1 and with other supplies the cost per cubic yard would be about $1.10. Comparing this method with the use of drills, the average charge used per cubic yard in drilled rock is 1 pound; the cost of blasting would therefore be about 55 cents per cubic yard. At the present scale of wages, figuring 40 feet per machine per day, drilling costs 87 cents per cubic yard on many of the forest roads on account of their inaccessibility and high transportation costs and the extreme hardness of much of the rock. The comparison can be readily made with this data; the cost is $1.10 without drilling and $1.42 if the rock is drilled. The use of TNT instead of drilling saves 32 cents. Under other conditions the saving would not be so great with TNT valued at 50 cents per pound.



THESE ROADS TO BE CLEARED OF SNOW

Notes: Beaver to Pittsburgh, Pa., practically continuous city street
Philadelphia to Delaware line, a toll road



ACCORDING TO STATE HIGHWAY DEPARTMENTS

THE HOW AND WHY OF TRUCK IMPACT

By Earl B. Smith, Senior Assistant Testing Engineer, Bureau of Public Roads.

THE impact forces brought to bear on modern highways by heavy motor trucks have for some time occupied a large share of the attention of highway engineers. The study of these forces, their amount and character and the more accurate determination of their effect on highway surfaces is one of the major subjects under investigation by the division of tests of the Bureau of Public Roads.

Previous articles in this magazine have reported the progress which has been made in determining the intensity of such forces, and later articles will enter more fully into this phase of the subject and add to the large body of test data which is rapidly developing. In addition to the intensity of the forces themselves, it is highly desirable to investigate their effect on the road surface. For the lack of a better method of expression the results of the tests conducted by this bureau have been recorded in terms of "equivalent static load." Though this will suffice for the present, it is very probable that some better measure of the effect of impact must be obtained before it will be possible to design roads in a thoroughly rational manner. That the effect of impact is not the same as the effect of equivalent static pressure is a matter of common observation. Everyone has observed that steady pressure applied to a pane of glass will splinter and break it in many directions, while a high velocity bullet directed against a similar pane will pass through it leaving a small hole in an otherwise uninjured pane. This effect is now receiving the attention it has long deserved, and the results thus far obtained in the tests which are under way encourage the belief that possibly we shall be able to account for the difference between impact and static pressure.

It is not the intention of this article to enter into a discussion of these subjects, but simply to present a brief analysis of the action of the various parts of the motor truck as it passes over the road, and of the effect of the several parts in producing impact and pressure forces which are applied to the road through the wheels.

FACTORS WHICH PRODUCE IMPACT.

The impact which results when the moving truck strikes an obstruction on the road or passes over such surface irregularities as waves, potholes, and joints, is a function of several factors, most of which originate in various parts of the truck. These factors are: The sprung weight, or all weights and loads above the springs; the unsprung weight, which includes wheels, tires, axle, springs, and all other effective weights under the spring; the kind and condition of the tire; the spring characteristic which is its deformation or deflection under different loads and its period of vibration; the horizontal speed of the truck; and the character of the road surface involving the size and shape of the obstructions or irregularities over which the wheel passes.

To deal first with the last-named factor, the irregularities and obstructions presented by the road surface must share responsibility with the irregularities and defects of the tires as the prime cause of whatever impact is produced. Given a perfectly smooth road surface, traveled by a truck with perfectly smooth circular tires, and there would be no impact, regardless of the truck's weight and speed. But since perfect smoothness of either road or tire is a condition it is practically impossible to attain, it follows that impact must be recognized as an inevitable force and its destructive effects may be reduced by proper design of the road and the truck.

In general, the effect of increasing the horizontal speed of the truck is to increase the impact. The belief formerly held that the impact would vary as the square of the speed is not borne out by careful investigation. In the tests made thus far it has been observed that the intensity of the impact has varied with some power of the speed, but this power is less than two and may be as low as one. Indeed, under some conditions, as when the truck is projected over the edge of a horizontal section of pavement and falls to a lower section the impact may be practically independent of speed.

The factors of speed and surface and tire irregularity may be regarded as the factors which create the conditions resulting in impact. The quality of the impact and the intensity of it are naturally affected by the relations of sprung and unsprung weight, the character of tires, and the characteristics of the spring used in the design of the truck itself.

Since impact is the force resulting from a mass being moved or brought to rest with a certain acceleration, it is greatest when "mass times acceleration" is a maximum. Whatever adds to mass or acceleration tends to increase impact and whatever is done to reduce mass or acceleration will tend to decrease impact.

THE EFFECT OF THE SPRINGS.

In dealing with motor trucks the total mass is really made up of two masses which are separated from each other by a spring. If the whole mass falls free from some considerable height (that is, several feet) its net acceleration will be that of gravity, but the negative acceleration at the moment of impact will not be the same as if the truck were a single solid mass. This is the result of the cushioning action of the springs which reduce the deceleration of the sprung parts. In this case the sprung parts really do not add to the impact of the unsprung parts, but rather produce intensified pressure a moment after impact when the spring deflection reaches the maximum. This action of the spring has simply reduced the deceleration of a part of the mass and thereby decreased the impact effect from what it would have been if the whole acted as a single rigid mass.

Now, let us suppose that the truck be supported at a height of only 3 inches above the road instead of several feet as in the former example, and suppose that the deflection of the spring under the weight of the part of the mass above it is greater than 3 inches. If, then, the whole mass is dropped by releasing that part of the mass under the spring this unsprung mass will be accelerated downward by the combined effect of gravity and spring pressure with the result that it will have a very high velocity and be brought to rest very suddenly upon striking the pavement. This downward velocity and final deceleration may be very much greater than could be obtained by a free fall from a height of several feet. In this case the sprung mass has dropped very slightly but the initial compression of the spring has increased the velocity of the unsprung parts and thereby increased the impact produced by those parts.

Again, suppose that instead of supporting the truck at a height above the road which is less than the spring deflection, it is supported at some height somewhat greater. Now, if the unsprung weight be suddenly released, the whole mass drops but the unsprung parts by virtue of the propulsion of the spring will at first be accelerated faster than the mass as a whole. Then if the drop is sufficient to permit the spring to reach the limit of its oscillation before the wheels strike the road, the effect will be to decrease the velocity of the unsprung parts, relative to the whole mass, by the reverse action of the spring. In this case the action of the spring may actually serve to reduce the impact of the unsprung parts.

Thus it is seen that the springs of a motor truck tend to reduce the impact of the truck as a whole by reducing the vertical deceleration of the sprung parts; that under some circumstances they may reduce the velocity of the unsprung parts, but that under most actual conditions they may tend to increase impact by adding to the velocity of the unsprung parts. Their effect upon the impact in any case, is through the modification of the deceleration of the sprung and unsprung portions; this effect on the unsprung portion being due to a change in the vertical velocity at the beginning of impact.

THE EFFECT OF THE TIRES.

Having analyzed the action of the spring in this way, let us now examine the effect of the tires. The tires are designed to do for the unsprung parts what the spring does for the sprung parts, i. e., to reduce their vertical deceleration. The extent to which they accomplish their purpose depends upon their ability to deform or yield under the blows of the truck above them. The harder they are the greater will be the acceleration or deceleration of the truck mass, and the less will be their effect in reducing impact. A wheel tired with hard rubber, upon striking an obstruction on the road surface will produce relatively great impact, because it is only slightly cushioned by the tire, and is brought to rest in a very short space of time. If the same wheel be fitted with a softer tire the impact will be smaller, and its value will be a function of the deformation of the tire. Herein lies the advantage of pneumatic over solid rubber tires.

THE TRUCK IN MOTION.

In the foregoing discussion the action of the springs and tires has been studied under conditions of free fall from an initial state of rest. The question which will naturally arise is, do these conditions pertain to the action of the truck when it is moving over the road? What actually happens when a truck moves over a road? In the first place, the wheels are continually rising and falling as they encounter obstructions or irregularities in the road surface. At the same time the tires are alternately in a state of compression and release. The unsprung parts follow the wheels, rising and falling more or less sharply according to the kind and condition of the tires. The motion of the unsprung parts is transmitted to the sprung parts through the spring which oscillates up and down under the repeated blows of the axles. Such is the effect of the springs, however, that the motions of the sprung parts are much less abrupt than those of the unsprung parts. Their rising and falling occupies a longer period, and because of this difference in the period of oscillation, at any instant the sprung and unsprung parts may be rising or falling together or the one may be rising while the other is falling.

The movement of the sprung parts when the wheel strikes an obstruction is comparatively slight at the instant of striking. Their period of vibration, because of inertia, is rather long in comparison to the period of the unsprung parts. The displacement or maximum movement of the sprung weight occurs at some time after the impact, and produces intensified road pressures, which may be greatly more than the total load of the truck.

The unsprung parts act in an entirely different manner. When the wheel strikes an obstruction in the road it acquires an upward velocity depending upon the height of the obstruction and the speed of the truck. The reaction on the road surface is a force depending for its value upon the maximum vertical acceleration of the wheel, the mass of the unsprung parts and the spring pressure. The wheel may continue to move upward even after passing the obstruction, until the movement is overcome by spring pressure and inertia. It is then shot downward by the combined action of gravity and spring pressure, and thus produces another impact on the road surface. The vertical reaction of this upward movement of the unsprung parts serves to retard slightly the fall of the body or sprung parts of the truck, and when this body does fall it is partially cushioned on the spring, and does not really produce an impact; but it does produce an additional pressure on the road surface somewhat like that of a suddenly applied load, which, as mentioned above, may be greatly more than the weight of the truck.

It will readily be seen that the magnitude of the impact as the wheel passes over obstructions is dependent upon the speed of the truck. It is the unsprung weight and the value of the spring pressure which produces the actual impact. The greater the spring pressure and the greater the unsprung weight the greater will be the impact. The movements of the sprung weight do not actually result in impacts, but in intensified pressures. As a factor in producing spring pressure, however, the sprung weight does add to the impact of the unsprung weight.

From these statements it may be seen that while the weight of the unsprung parts is a direct factor in the resulting impacts on road surfaces, it is not necessarily true that the heavier unsprung weights produce always the greatest impact. On the contrary, conditions may be such, with a light unsprung weight, on hard, nonresilient tires, under the influence of large accelerations or decelerations, that high impact values will be produced. It should be noted that this discussion of the effect of unsprung weights applies only to impacts on road surfaces and not to the effects produced on the truck itself.

THE EFFECT OF MASS.

Up to this point the attention has been centered upon the parts of the truck which affect impact through their effect upon the "acceleration" factor. The springs and tires reduce impact by reducing acceleration. It is obvious that the only other means of reducing impact is to reduce the mass of the truck, or since the impact is delivered through four wheels, to reduce the mass on any wheel or pair of wheels. In this direction lies the possibility of a considerable further reduction of impact, because motor trucks, as now designed, generally carry the larger part (up to 90 per cent) of their weight upon the rear wheels. In the average heavy truck the body is nearly balanced upon the rear axle. Normally a small amount of the carried load goes to the front wheels, but so near is the balance on the rear axle, that a slight eccentricity of loading will often result in decreasing the load on the front wheels. The effect of the truck upon the road surface depends upon the impact delivered by its heaviest wheel. It follows, therefore, that a substantial reduction in the destructive effect of the truck can be made by distributing the load more uniformly to the front and rear axles, thus reducing the load on the rear wheels.

This change alone would go far to reduce the heavy burden laid upon the public in the shape of repair bills for roads damaged by motor trucks. What else can be done? Roads can be kept smoother by better maintenance, though in the nature of the case they can not be kept so smooth as to eliminate impact entirely. Solid rubber tires can be kept in better repair, or better still, abandoned in favor of cushion or pneumatic tires wherever possible. Operating speeds can be kept within reasonable limits. Such measures will reduce the intensity of the destructive force. Whatever impact remains when these things have been done is the force which it is the duty of highway engineers to provide against in the design of future highways.

SUPERELEVATION AND EASEMENT AS APPLIED TO HIGHWAY CURVES

[Concluded from page 12.]

ence is in the State of New York where an additional width of 3 feet is allowed on curves having a radius of 2,500 feet, which additional width is increased by stages until for a radius of 300 feet an additional width of 8 feet is used. Unquestionably this adds something to the safety with which curves can be used by fast moving vehicles, but it also adds considerably to the cost of construction and under ordinary circumstances, that is, where both safety and expense must be considered, a combination of the New York standards as to the amount of superelevation and the West Virginia standards as to widening with a liberal allowance for transition distances will probably prove to be as safe as any solution which can be devised and as cheap.

SUPERELEVATION OF REVERSE CURVES.

Ohio specifies that, where possible, reverse curves shall be separated by tangents at least 100 feet in length. On short curves, where this is impossible, the cross-section is made level at the point of reverse curve and the superelevation gained in a distance of 100 feet in each direction. Mr. G. A. Curtis, district engineer of the Massachusetts Highway Commission, suggests that 30 feet of transition should be allowed for each one-fourth inch change in pitch. Thus the change from one-half inch pitch per foot of width to a full reverse of one-half inch pitch per foot of width would require the same distance as to reach a superelevation one inch per foot of width, that is, 120 feet. Where parabolic curves or spiral transition curves are used an intervening tangent, though desirable, is not as necessary as it is in most of the prevailing designs.

DESIGN AND CONSTRUCTION OF BRIDGE FOUNDATIONS

By Llewellyn N. Edwards, Senior Highway Bridge Engineer, Bureau of Public Roads.

IN view of the rapidly increasing demands of traffic upon our highways the design and construction of new bridges should involve provisions not only for existing conditions but also for those which may reasonably be expected in the future. Naturally enough we can not plan intelligently for the future unless we can foresee the conditions with which we will have to deal; we can not create efficient designs unless we have a knowledge of the future demands as to capacity and sustaining power. However, all will agree that careful surveys of service requirements both immediate and future will assist in the making of well developed guesses.

REINFORCED CONCRETE ARCH BRIDGE WRECKED BY STREAM SCOUR. AS CONSTRUCTED, BRIDGE HAD SHALLOW FOUNDATION ON GRAVEL. WATERWAY APPROXIMATELY 50 PER CENT THAT REQUIRED.

Speaking generally, a bridge structure consists of three portions or parts, viz:

(1) The superstructure or spanning portion supporting, in addition to its own weight, pedestrian, vehicular, and other so-called "live" loads.

(2) The substructure supporting the superstructure with its loads, and,

(3) The foundation which supports the substructure with its superimposed loads.

Obviously the stability of the entire structure depends primarily upon the efficiency of its foundation. However, it may be said without fear of contradication that science, skill, and experience combined with the progress of the time have achieved marked success in evolving the rationale of bridge design and in producing and developing new types of superstructures and substructures, while a far less degree of attention has been bestowed upon the development of fundamental principles and reliable information and data relating to the bearing power of soils, and to methods and operations involved in the design and construction of foundations.

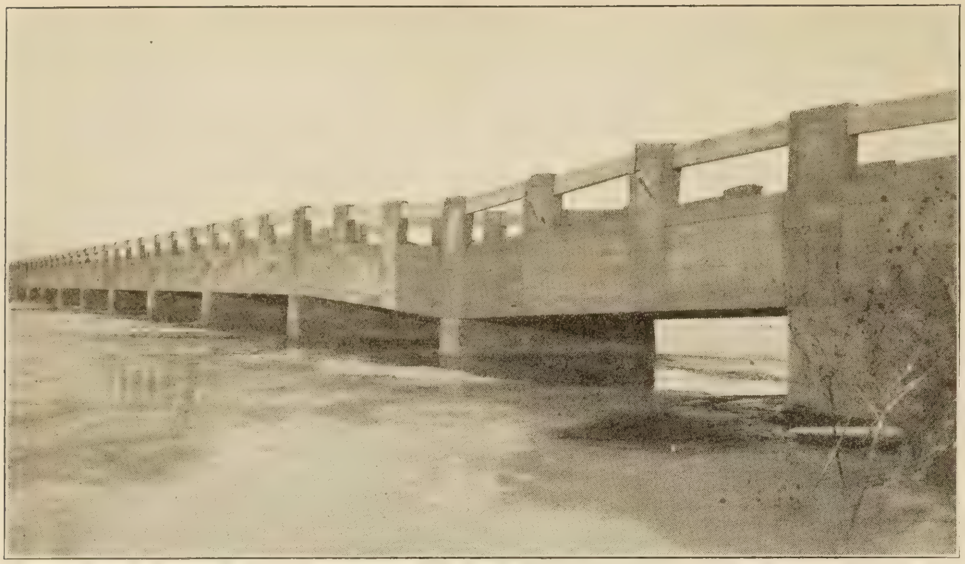
In this connection it is especially important to note that a very large percentage (probably 70 per cent or more) of bridge failures are primarily due to unsatisfactory foundation conditions. The money value involved in these failures is, to say the least, unfortunate but quite apart from this, the inconvenience and economic disadvantages incident to the tying up of traffic by failures, and the moral responsibility attached to the construction and maintenance of structures menacing human life demand full consideration.

CONDITIONS AFFECTING STREAM SCOUR.

With an occasional exception, the construction of bridges over streams has the effect of obstructing free stream flow by contracting the channels and by producing eddies, cross currents, whirlpools, etc., resulting in accentuating any tendency that may exist to erode or scour the beds during periods of high water. The location and extent of scour is mainly dependent upon the courses of the streams, the rate of their currents, and the character of the soils forming their beds. The greatest scour will generally take place in the vicinity of the portions of the structure furnishing greatest resistance to the free flow of the water. In general, piers and abutments should have their foundations located well below the range of possible scour. However, to render them secure against unforeseen eventualities they are often protected by the installation of riprap, stone filled cribs, brush mattresses weighted down with large stones, concrete aprons, etc.

ADEQUATE FOUNDATION SURVEYS IMPORTANT.

In general, the loads considered by the designer, in proportioning bridge pier and abutment footings are (1) the total dead load of the superstructure and substructure, (2) the live load including under certain



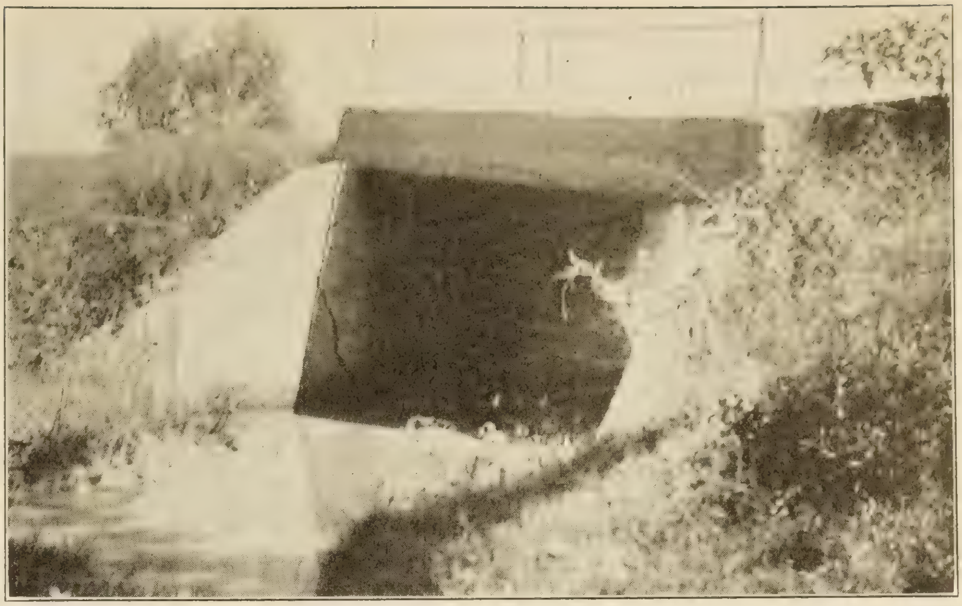
PIER FAILURE RESULTING FROM STREAM SCOUR. SAND OF RIVER BED "ALIVE" AT HIGH-WATER STAGES. PILES IN FOUNDATION HAD INSUFFICIENT PENETRATION.

conditions an allowance for impact, and (3) earth pressures in the case of abutments acting as retaining walls. To the foregoing may be added (4) forces induced by temperature in the case of piers and abutments for arch spans, and (5) wind, ice, or other external pressures in exceptional cases. In the absence of definite data as to the bearing value of the foundation this refinement of load calculation would be of no advantage. It is therefore of basic importance that the designer be in possession of all available information relating to foundation conditions. The securing of this information necessarily involves a survey of soil conditions by making borings, sinking test pits, driving test piles, etc., to discover subsurface conditions. It also involves a general survey to obtain data relating to the size of the drainage area, rate of stream flow, direction of stream currents, high and low water elevations, possibility of accumulations of drift, etc., likely to affect stream scour, together with a contour survey of the bridge site.

The general formation, density, and thickness of strata of soil vary so much that the testing of the foundation soil and the depth of underlying strata are absolutely necessary for the determination of the allowable pressures that may be placed upon the foundation material. The course, depth, and velocity of the stream in the vicinity of the bridge site, together with natural conditions influencing the future development of surrounding property, should receive full consideration in their relation to stream currents and scour of the stream bed.

Too frequently survey plans provide insufficient information for the preparation of the design and the working drawings of bridge structures. The character of the foundation materials is too often assumed to be the same as that showing upon the surface at the bridge site. Occasionally the superficial examination of the soil is primarily due to the fact that the equipment of the survey party does not even include a mud auger with which to make the simplest kind of a subsurface examination. However, there are other factors which sometimes enter into the field survey work. The observer, not realizing the importance of accurately describing the character of the foundation soil, fails to incorporate a proper description in his field notes; or, what is even worse, the draftsman preparing the survey plan, believing that "brevity, brevity always" is a mark of efficiency, will eliminate all tendencies to verbosity and will concentrate paragraphs into single words such as "clay," "gravel," "bowlders," etc.

To emphasize the importance of the foregoing it may be of value to cite an actual case of construction work. The bridge referred to was located at the outlet of a comparatively large lake. The survey party, although equipped with a mud auger outfit, made only a surface examination of the soil and the survey plan prepared from its field notes showed "bowlders with clay" as foundation material. "Bowlders with clay" constitute under certain conditions excellent foundation material. However, in this particular case the "bowlders" were merely "one man" stones which had been deposited along the river banks by ice floes. Knowing nothing of the field conditions other than was shown upon the survey plan the designer assumed a foundation pressure which seemed justified by the description and designed the substructure accordingly, placing the foundation elevation at a depth below possible frost action. When the foundation pits had been excavated to a depth of 2 to 3 feet all of the "one man" stones had been removed and the underlying blue clay was soft and altogether unreliable as a foundation material. Soundings were made with a steel bar by one of the workmen which indicated the existence of firm material at a depth of approximately 3 feet below the foundation elevation shown on the substructure plan. When the overlying clay had been removed a hard, durable limestone foundation was secured. This material was capable of safely sustaining a much greater load per unit area than had been assumed in the original design and had conditions permitted, a considerable economy could, doubtless, have been affected by redesigning the entire substructure.



REINFORCED CONCRETE SLAB BRIDGE FAILURE. ALLUVIAL SOIL FOUNDATION MATERIAL. FOUNDATION INSUFFICIENT DEPTH BELOW STREAM BED.

PRESSURE DISTRIBUTION NOT ALWAYS UNIFORM.

Having discovered the character of the foundation material by a careful survey, it is important to remember that foundation pressures are not always uniform. Other conditions being equal, a pier symmetrically loaded and symmetrically shaped will produce a uniformly distributed pressure upon its foundation, but irregularities in the lengths of superstructure spans, cut water construction on the upstream end, subjection to heavy ice floes, etc., tend to produce variable foundation pressures. Abutments are commonly so located as to perform the double function of supporting the superstructure and resisting the pressure of the earth fills forming the bridge approaches. The pressure exerted upon an abutment foundation is, therefore, rarely a uniform one and in consequence, the so-called "toe pressure" is greater per square foot of foundation area than is the "heel pressure."

The pressure per square foot used for designing purposes should be well within the safe bearing power of the soil; that is, a pressure which will not produce appreciable settlement. If, perchance, an undue settlement takes place, the abutment will tilt forward and may ultimately result in a total failure of the structure.

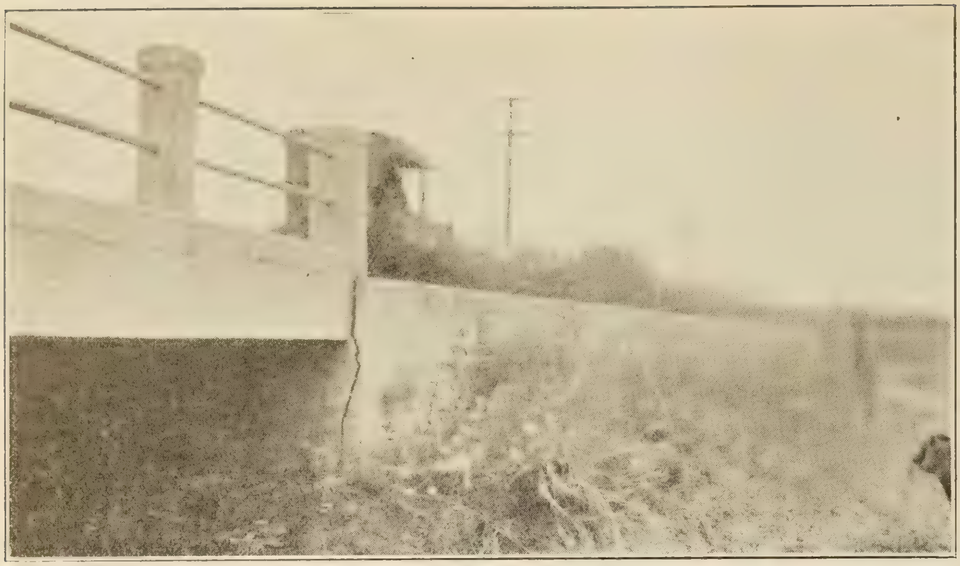
One of the distinctive advantages resulting from the common practice of building piers with battered sides and ends and abutments with battered face surfaces on the body and wings is that a slight tilting may take place without giving to the substructure an overhanging and unstable appearance. It is important in this connection to call attention to certain structural weaknesses resulting from substructure and foundation movements. Almost without exception such movements subject the superstructure to stresses for which no provision was made in its design, though the actual condition existing may not become fully apparent even by the most careful and critical examination. A pin-connected truss span gives greater evidence of the conditions mentioned than does a riveted span. By reason of a slight derangement of the correct alignment of the bottom chord pins, a portion of the eye bars become loosened, thereby rendering them partially or entirely inactive with the natural result that the stresses these bars were designed to carry are actually carried by the other bars. Structures composed of mass and reinforced concrete frequently give evidence of substructure and foundation movements, for in these structures the induced stresses appear to seek out the weakest portions and, in consequence, the resulting fractures may take most unfortunate courses and by progressive action may result in the development of very dangerous structures or possibly in failures.

Speaking generally, foundations are either natural soils, including rock formations, or artificial foundations consisting of compacted soils, soils reinforced with piles, grillages, etc.

NATURAL FOUNDATION MATERIALS.

Natural soils are an ever varying quantity, heterogeneous in their composition and stability. Their bearing power depends mainly upon the general formation, texture, density, uniformity, and thickness of the strata. A stratum composed of a soil having, under ordinary conditions, a comparatively low bearing power may be so confined by a thick, strong overlying stratum that it will sustain with ample safety a greatly increased load. A similarly placed stratum located with an outcropping in the hillsides of a deep ravine or river valley may be subject to a very slow oozing flow under the pressure of a superimposed abutment or pier, which will ultimately result in a sufficient settlement of the foundation area to produce appreciable superstructure deformations. Usually under these conditions the vertical movement is combined with a lesser movement of the overlying material, producing a horizontal displacement of the bridge structure. Such movements may involve a period of months or even of years to become fully developed. They may continue at a practically uniform rate or may become accelerated by reason of a weakening of the overlying strata resulting from fissures, fault lines, etc., existing within them.

Natural soil foundation materials are divided into five general classes, viz: (1) Bedrock, (2) gravels,



OVERTURNING RETAINING WALL. EARTH PRESSURES ON HIGHWAY SIDE OF WALL GREATER THAN ITS RESISTANCE TO OVERTURNING.

(3) sands, (4) clays, and (5) ordinary soft soils and earths.

BEDROCK AS A FOUNDATION.

Bed rock is, as a general rule, practically secure against stream scour and under ordinary working pressures is incompressible. However, its surface may be irregular, presenting shelly areas and numerous cracks and fissures, through which water frequently flows very freely, rendering the work of preparing foundation areas costly and difficult. On the contrary, it may present a surface worn smooth by glacial or water action and rounded or tilted at such an angle with the horizontal that it becomes necessary to level it horizontally as a whole or in steps in order to render the substructure, piers, or abutments placed upon it secure against scour, sliding, etc.

The influence of the elements upon bedrock must be taken into consideration, especially when outcroppings are contemplated for use as foundations, as certain rock formations are subject to disintegration when so exposed.

The brief discussion of bedrock materials here given would be incomplete without a reference to that grade of soils which, for want of a better nomenclature, will here be referred to as "near rock." It is especially important to exercise caution whenever these materials are being considered for foundation purposes. The most common of these are:

(*a*) Soft shale, sandstone, limestone, and other rocklike material generally known as "rotten rock,"

(*b*) Very compact and shale-like beds of clay which are subject to rapid disintegration when subjected to scour.

(*c*) Beds of compact clay, similar to the above, which tend to slake when subjected to atmospheric conditions.

(*d*) Beds of cemented gravel or sand which disintegrate freely when subjected to scour.

As a general rule, "near rock" soils are appreciably compressible when submitted to fairly high unit pressures and, in addition, many of them disintegrate freely when subjected to the erosive action of swift currents.

GRAVEL A GOOD FOUNDATION.

Gravel varies widely in its granulometric composition and in the original rock material of which it is composed. Gravel is not readily defined since its many uses in bridge, road, and other construction work have established for it various limitations as to maximum and minimum sizes of particles. However, a somewhat commonly accepted definition is: "Small worn fragments of rock material which will pass through a screen having holes $2\frac{1}{2}$ inches ($2\frac{1}{2}$-inch screen) in diameter and be retained upon a sieve having four meshes per linear inch (No. 4 sieve)." In so far as its use as a foundation material is concerned, the term "gravel" is very generally applied to material having approximately the above-described grading intermixed with sand.

A gravel having its particles well graded from coarse to fine and firmly cemented with a natural earthy cement containing iron, silica, lime, or other comparatively stable cementing material, if existing in a stratum 4 to 8 feet or more in thickness, is practically incompressible under the pressure produced by ordinary bridge structures and provides an efficient foundation even when overlying a less reliable substratum. Well cemented gravel will withstand the disintegrating influence of a quite rapid current. Slow disintegration may sometimes result from dissolution of the cementing medium. The cemented gravels here referred to must not be construed to mean the hardest grades of conglomerates which quite properly may be considered as bed rock, but, instead, the softer conglomerates which can ordinarily be broken up in an excavation by the use of a pickax.

A confined gravel stratum made up of loose but well compacted material, free from the disintegrating action of water flowing through it is only slightly compressible under ordinary foundation pressures.

The illustration on page 19 shows the result of building on a shallow gravel foundation combined with a structure too short in length to serve stream requirements.

FOUNDATIONS IN SAND.

Sand, like gravel, varies widely in its granulometric composition, in its petrography, and in its other physical properties tending to render it satisfactory or unsatisfactory as a foundation material. It may exist as a firm, well cemented, granular material practically incompressible under favorable conditions or, on the contrary, it may, when fine grained and saturated with water, exist as a soft, yielding, semifluid material quite unreliable for foundation purposes. As in the case of cemented gravel, above described, the cementing medium is commonly a natural earthy cement containing iron, silica, lime, or other cementitious material. Special care must be exercised whenever the cementing material is readily soluble in water, since its subjection to the action of flowing water would cause a disintegration of the foundation.

FAILURE OF AN ABUTMENT SUPPORTING A WOODEN SUPERSTRUCTURE. INSUFFICIENTLY DESIGNED ABUTMENT. FAILURE RESULTED PRIMARILY FROM SCOURING OF FOUNDATION.

Compact sand, thoroughly confined against lateral movement by overlying strata or otherwise, and existing in beds 6 to 10 feet in thickness, is considered an excellent foundation material, provided its stability is not endangered by a free flow of water through it. "Quicksand when properly drained makes a very stable earth, but when subjected to saturating water it becomes a hopeless mess."

Sand originally deposited by the action of water commonly exists in layers or strata varying widely in their granulometric composition. Quite frequently these strata have been deformed by subsequent movements so that they are tilted at an angle with the horizontal. A firm, well compacted stratum so tilted may be rendered useless for foundation purposes by an unreliable substratum containing an unstable loose sand, clay, or other unsatisfactory material which will fault when subjetced to foundation pressures. On page 20 is shown a pier failure resulting from a too shallow penetration of piles in a river bed composed of sand.

CLAY AN UNCERTAIN MATERIAL.

Clay may exist as a firm, unyielding material resembling shale rock and having both cohesion and density, or on the contrary it may exist as a pasty, semifluid material having very little supporting power. This range of variation in its physical composition renders it one of the most uncertain and unreliable of foundation materials. Speaking generally, the best clay deposits of the former class are quite impervious to water and when protected from the disintegrating action of flowing water are quite reliable as foundation materials; however, it may here be remarked that some of the most dense clays are very readily scoured; also that clays giving fair evidence of stability will very frequently, through absorption of water, become less stable and compact, thus losing the important elements of a good foundation material. Not infrequently clays are encountered which, when subjected to foundation pressures, will squeeze and ooze like a firm India rubber.

"Clay even if found in hard condition is subject to slight compression, which, while uniform, may not be of much consequence." However, mention has been made above that the foundation pressures produced by abutments and retaining walls are very commonly variable from front to rear of wall, thereby tending to produce an uneven settlement. Drainage of clay foundation areas will increase their stability and bearing power.

Clay strata which have been deformed and are tilted at an angle with the horizontal are subject to possible slipping along natural cleavage planes rendering them to a greater or less degree unreliable as foundation materials. Their dependability in such cases is based mainly upon the strike angle of the formation and the physical character of the clay strata composing the general mass. In this connection, it is important to call attention to the fact that clay strata frequently contain fissures and fault lines running at random through them. An eroded hillside may expose these planes of natural weakness in such a manner as to render a hillside foundation of clay material rather uncertain as to its effectiveness.

Ordinary soft soils and earths, when considered as foundation materials, vary in relation to the proportions of sand, clay, or other firm materials they contain; however, in general these soils will support only very small foundation pressures. They are always subject to a considerable amount of settlement and should, therefore, be used only under the most unimportant structures with comparatively small pressures per unit of foundation area. Soils containing materials which render them plastic and greasy are considered very poor foundation material. On page 21 is shown a reinforced concrete bridge failure resulting from a too shallow depth of abutment foundations in an alluvial soil. Settlement was produced by undermining of both abutments.

With the exception of bedrock, nearly all soils are appreciably compressible when subjected to ordinary foundation pressures. The amount of settlement depends upon the character of the material and the intensity of the pressure upon it. Usually the latter is expressed in tons per square foot of foundation area. Decisions as to the adaptability of natural foundation materials and the pressures they will safely sustain without undue settlement, demand mature judgment which has been ripened by personal experience and a knowledge of the results obtained by others. The amount of settlement permissible for any given case depends, to a marked degree, upon the character of the design and of the materials used in the bridge superstructure. The permissible settlement is, for certain classes of structures, the gauge by which the allowable unit foundation pressures must be measured and final decision rendered. On page 22 is shown a retaining wall "turning turtle" either as a result of improper design or excessive "toe" pressure on its foundation.

Reference has been made to the paucity of reliable data relating to the bearing power of soils. In the absence of this data we must be guided almost entirely by past experience. Safe foundation pressures for natural soils are given in Table 1. To obtain good results the use of this table should be supplemented by an examination of the foundation material in the abutment and pier pits. Otherwise its use will occasionally lead to unsatisfactory results since any table of this kind can not possibly take full account of widely varying soil conditions.

Table 1.—*Safe bearing power of foundation soils for ordinary structures.*[1]

Material.	Safe bearing power, in tons per square foot. Minimum.	Safe bearing power, in tons per square foot. Maximum.
Rock, the hardest kinds, in thick natural beds	(2)	(2)
Granites, limestones, and sandstones, hard grades	30	40
Granites, limestones, and sandstones, etc., medium hard grades	20	30
Limestones, sandstones, shales, etc., equal to best hard-burned brick masonry	15	20
Limestones, sandstones, shales, etc., equal to ordinary brick masonry	5	10
Gravel or coarse sand, in thick beds, well cemented	8	10
Sand, medium, in thick beds, well cemented	4	6
Sand, clean and well drained, confined	2	4
Sand, clean and wet, confined	½	3
Sand and clay combined, in thick beds, well drained, confined	5	8
Clay, firm, in thick beds, well drained, confined	4	6
Clay, firm, in thick beds, wet but not saturated	2	4
Clay, soft, confined	1	2
Alluvial soils, firm	½	1

[1] Arches, cantilevers, continuous spans, large structures, and high abutments should be founded on unyielding material. Special investigations and designs should be made for such structures.

[2] Any ordinary substructure load.

ARTIFICIAL FOUNDATIONS.

Artificial foundation materials are divided into four general classes, viz: (1) Compacted soils, (2) grillage, (3) piles, and (4) cribs.

Compacted soil foundations are sometimes used for small drainage structures having comparatively low foundation pressures per square foot. They are produced by treating soft soils and earths to render them more firm and more capable of supporting foundation pressures. Two methods of compacting are in use, viz: (1) A thick, thoroughly compacted bed of sand, gravel, or broken stone is deposited upon the soft soil after excavating it to a considerable depth. (2) A quantity of sand, gravel, broken stone or concrete is forced into the soft soil.

Grillage, as commonly constructed, consists of two or more courses of wooden timbers, laid transversely to each other and firmly drift-bolted together. The timbers may be laid in close contact or they may be separated and additional strength secured by filling the spaces between them with sand, gravel or concrete. Grillage may also be composed of metal beams filled in with concrete or of concrete reinforced by rods or other metal work to give it the desired strength. In order to secure a desired foundation pressure the grillage area is usually much larger than the base area of the substructure unit placed upon it.

Piles are more commonly used than any other class or type of artificial foundation material. Their use during centuries past has proved their reliability, especially in soil foundations where excavation to a firm unyielding stratum is unattainable by reason of excessive cost or other practical considerations. Formerly wooden piles were used exclusively, but in recent years reinforced concrete piles have been used to a considerable extent. The types, sizes, methods of preparation, etc., of piles, their proper driving, and considerations entering into the determination of their safe bearing power, although directly related to our general subject of foundations, can not be satisfactorily treated within the space here available. However, it is important to call attention to some common misconceptions of the bearing power of piles.

PILES IN SOFT SOIL.

A pile driven its entire length through soft soil depends for its bearing power upon the friction existing between its surface and the soil surrounding it. This so-called skin friction transfers or imparts to the surrounding soil practically the entire load carried by the pile; the resistance of the soil under its point and the effect of buoyancy are but small in comparison with the load transmitted to it by the substructure. On account of this fact it is improper to drive piles in soft soils at close intervals. A cluster of, say, seven piles driven in close contact with each other can scarcely be considered as developing a bearing power in excess of that developed by three piles properly spaced to transmit their loads to the surrounding soil.

Piles, dependent wholly or in part upon skin friction, produce, by transmitting their load to the surrounding soil, a corresponding soil settlement. It follows, therefore, that the soil underlying the base of the substructure may, in assuming its load, settle out of close contact with the substructure base, thus becoming independent of any foundation pressure apart from that transmitted by the piles.

By a closer spacing of the piles under the toe of an abutment than under the heel it is possible to prevent the tipping of the abutment which would otherwise take place if settlement should occur. By varying the spacing a practically uniform load per pile is secured. If perchance a future settlement of the abutment should occur, this arrangement of the piles would tend to make it uniform.

Generally speaking, engineers do not consider it satisfactory to count upon the transverse strength of a piled foundation as tending to resist the horizontal thrust of arches or other structures producing pressures acting transversely to the direction of penetration of the piles. However, it is occasionally argued that the soil upon the sides of the piles opposite the direction of the pressure can be relied upon to resist that pressure. Granting that this contention is in part true, the question arises as to the extent to which this soil can reasonably be considered as acting. At best, the problem is a complex, indeterminate one. It involves such factors as: (1) To what depth below the base of the substructure shall the thrust be considered as resisted by the piles, the soil being compressible and the piles flexible? (2) Assuming the piles in adjacent rows to be staggered, are more than the two rows of piles on the side of the foundation area opposite to the direction of the thrust active in resisting it? Naturally, any pressure borne primarily by the other rows of piles will be transmitted with little, if any, loss through the intervening soil to the two rows above mentioned. (3) To what extent is the tendency of the transverse thrust effective in disturbing the uniform distribution of the skin friction element on all piles in the foundation? Considering the indeterminate nature of these problems, and considering further that a very slight movement of the superstructure upon its foundation will induce important stresses never considered in the original design, it seems wise to forego all snap judgment and to introduce battered piles in the foundation, even at a considerable increase in cost per linear foot of the piles so driven.

THE USE OF CRIBS.

Cribs, as generally constructed, consist of grillage bottom frames with box-shaped sides extending upward from them. They are very commonly built at or near the bridge site, launched and towed out to the positions where they are to be sunk. If for a given pier, the final foundation area is covered by a layer of soft soil, it may be dredged prior to placing the crib in position or, in lieu of this, the crib may be sunk upon the soft soil which may be removed by clamshell dredging or by pumping it through the openings in the grillage bottom of the crib. If the foundation area be composed of bedrock, it may be necessary to predetermine its contour and to shape the bottom of the crib roughly to fit it, thus making possible the sealing of the crib against the admission of water preparatory to unwatering it for the construction of the pier within it.

A type of construction sometimes used consists of a piled foundation topped with a wooden crib within which the substructure, pier, or abutment, as the case may be, is built. The piles after being driven are cut off at a uniform elevation that will bring the top of the crib below minimum low water and the crib is floated into its correct position and sunk upon them. Under certain conditions this type of construction is both efficient and economical.

FEDERAL AID ALLOWANCES

PROJECT STATEMENTS APPROVED IN OCTOBER, 1920.

State.	Project No.	County.	Length in miles.	Type of construction.	Project agreement signed.	Estimated cost.	Federal aid.
Alabama	62	Talladega	23.000	Chert	Oct. 21	$332,472.03	$166,236.01
	77	Chambers	16.800	Top soil	Oct. 13	127,762.25	63,881.12
	81	Mobile	14.000	Gravel	Oct. 25	214,898.72	107,449.36
	83	Morgan	10.100	Macadam	Oct. 11	298,505.35	149,252.67
	88	Marshall	18.159	do	Oct. 16	563,266.55	281,633.27
	89	Geneva	24.190	Sand-clay	Oct. 11	170,068.25	85,034.12
	90	Coosa and Tallapoosa	13.500	Macadam	Oct. 23	367,771.25	183,885.62
	92	Macon	20.500	Gravel	Oct. 25	379,773.35	189,886.67
Arizona	41	Maricopa	6.473	Concrete	Oct. 21	200,000.24	100,000.12
Arkansas	53	Crittenden	[1] 13.730	Gravel macadam	Oct. 12	[1] 107,975.89	[1] 30,000.00
	90	Nevada	[1] 16.360	Gravel	Oct. 25	[1] 143,280.55	[1] 68,000.00
Colorado[1]	8	Rio Blanco	[1] 17.500	Grading and draining	Oct. 26	[1] 21,918.40	[1] 10,958.20
	55	El Paso	6.629	Sand-clay	Oct. 25	66,048.07	33,024.03
	97	Powers	6.669	Gravel	Oct. 23	82,647.84	41,323.92
	124	Conejos		Bridge	Oct. 25	14,987.00	7,491.00
Georgia	179	Green	2.700	Sand-clay	Oct. 13	16,529.81	5,000.00
	182	Jenkins	8.180	Sand-clay and gravel	Oct. 27	65,587.50	30,000.00
	190	Murray	2.700	Top soil and bridge	Oct. 13	20,075.00	10,037.50
	192	Bibb	2.272	Sand-clay	Oct. 21	55,941.72	5,000.00
	197	Bullock	2.250	Hard surface	Oct. 13	58,356.65	27,000.00

[1] Withdrawn or canceled.

PROJECT STATEMENTS APPROVED IN OCTOBER, 1920—Continued.

State.	Project No.	County.	Length in miles.	Type of construction.	Project agreement signed.	Estimated cost.	Federal aid.
Illinois	17	Adams	14.390	Earth	Oct. 16	$253,737.99	$63,434.49
Kansas	20	Johnson	[1] 6.000	Bituminous macadam	Oct. 12	[1] 65,033.32	[1] 9,754.99
	36	Cherokee	[1] 7.000	Concrete	do	[1] 258,656.20	[1] 105,000.00
	71	do	3.080	do	Oct. 11	337,700.00	46,200.00
	72	Atchison	3.125	Earth	Oct. 27	26,400.00	6,250.00
Kentucky	43	Nelson	3.800	Bituminous	Oct. 16	110,880.00	55,440.00
	44	Trigg	10.500	Waterbound macadam	do	200,200.00	100,100.00
Louisiana	63	Avoyelles	31.740	Gravel	Oct. 2	647,398.24	200,000.00
	77	Rapides	7.840	do	Oct. 18	98,382.62	49,191.31
Massachusetts	46	Worcester	4.261	Concrete	Oct. 13	196,438.00	85,220.00
Michigan	51	Sanilac	9.304	Gravel	Oct. 26	146,863.75	73,431.87
Minnesota	22	Dakota	17.040	Earth	Oct. 18	181,345.45	90,672.72
	79	Hubbard		Gravel	Oct. 16		[2] 4,063.05
	80	Polk		do	do		[2] 24,357.40
	82	Pipestone		Concrete	do		[2] 89,420.00
	86	Aitken		Gravel	do		[2] 5,684.50
	89	Le Sueur		Concrete, brick, or asphaltic	do		[2] 86,891.85
	93	Scott	7.130	Gravel	Oct. 12	154,993.16	50,000.00
	94	Polk		do	Oct. 16		[2] 22,135.53
	97	Olmstead		Concrete, brick, or asphaltic	do		[2] 75,397.44
	99	Todd		Gravel	do		[2] 12,492.70
	109	Redwood		do	do		[2] 3,275.00
	113	Crow Wing		Concrete, brick, or asphaltic	do		[2] 3,789.30
	121	Freeborn		Gravel	do		[2] 22,168.25
	122	Chippewa		do	do		[2] 12,395.91
	126	Rice		Concrete, brick, or asphaltic	do		[2] 45,776.04
	128	Chippewa		Gravel	do		[2] 16,960.00
	130	Big Stone		do	do		[2] 23,739.56
	136	Beltrami	2.940	do	Oct. 11	13,261.60	5,000.00
	148	Hennepin and Dakota		do	Oct. 16		[2] 59,116.54
	149	Aitkin		do	do		[2] 4,002.00
	150	do	5.840	do	Oct. 13	36,5471.60	10,000.00
	155	Polk		do	Oct. 16		[2] 34,423.67
	157	Itasca		Concrete, brick, or asphaltic	do		[2] 10,513.10
	161	Wilkins	18.780	Gravel	Oct. 13	195,408.40	97,704.20
	162	Beltrami	12.000	do	do	60,060.00	20,000.00
	164	Traverse		do	Oct. 16		[2] 15,012.10
	167	Hubbard		do	do		[2] 23,465.36
	169	do		do	do		[2] 5,434.70
	170	Mille Lacs	17.160	do	Oct. 12	145,459.60	72,729.80
	171	St. Louis	1.060	Concrete, brick, or asphalt	Oct. 13	105,227.21	21,200.00
	174	Dodge		Gravel	Oct. 16		[2] 13,097.78
	175	Cass	11.250	do	Oct. 13	122,100.00	45,000.00
	176	Itasca	7.430	Clay and gravel	Oct. 11	47,004.15	10,000.00
	179	Roseau	11.050	Gravel	Oct. 12	89,628.00	35 000.00
	181	St. Louis	1.070	Concrete, brick, or asphaltic	Oct. 13	57,527.36	21,400.00
	184	Lyon	22.830	Gravel	do	196,006.80	29,000.00
	185	Jackson		do	Oct. 16		[2] 4,255.00
	188	Freeborn	4.930	do	Oct. 11	35,565.20	3,000.00
	192	Hennepin	5.490	Earth	Oct. 13	36,746.76	10,100.00
	194	Lac qui Parle	10.370	Gravel	Oct. 12	29,564.70	10,000.00
	195	do	8.030	do	do	21,199.20	10,000.00
	196	Martin	7.000	do	do	78,639.88	25,000.00
	197	Dakota	.910	Brick, concrete, or asphaltic	do	54,883.22	18,200.00
	198	Lac qui Parle	5.780	Gravel	do	17,166.60	5,000.00
	199	Le Sueur	9.060	do	do	81,813.60	39,072.84
Mississippi	50	Warren	5.750	Concrete	Oct. 25	296,972.50	100,000.00
	100	Washington	10.000	do	Oct. 16	335,585.25	100,000.00
	101	Lauderdale	8.441	Gravel	Oct. 26	97,979.75	48,989.87
	106	do	3.040	do	Oct. 25	28,981.48	14,490.74
Missouri	159	Dade	24.500	Gravel and macadam	Oct. 11	94,399.31	47,199.65
	161	Stone	49.000	Earth	do	137,855.00	68,927.50
	165	Daviess	7.6 0	do	do	41,634.56	20,817.28
Montana	17	Flathead	9.000	do	Oct. 13	[1] 17,600.00	[1] 8,800.00
	102	do	[3] 1.600	Gravel	Oct. 11	[3] 165.00	[3] 82.50
	112	Fergus	.470	Concrete	Oct. 8	48,964.63	13,650.00
	136	Teton	13.200	Gravel	Oct. 11	76,780.00	38,390.00
	141	Missoula		Bridge	do	83,807.90	41,903.95
	142	Roosevelt	5.000	Gravel and earth	do	26,675.00	13,337.50
	144	Valley	18.000	Gravel	Oct. 13	118,855.00	59,427.50
	145	Lincoln	35.000	Earth	Oct. 1	224,290.00	112,145.00
	148	Valley	12.000	Gravel	Oct. 18	80,300.00	40,150.00
Nebraska	156	Hamilton and Merrick	26.800	Earth	Oct. 11	104,544.00	52,272.00
	164	Thayer and Nicholls	28.700	do	Oct. 21	128,144.50	64,072.25
New Hampshire	132	Strafford	.420	Gravel	Oct. 11	10,000.00	5,000.00
New York	75	Green	1.700	Concrete	Oct. 26	96,900.00	33,915.00
	80	Warren and Essex	8.300	Macadam	Oct. 11	473,100.00	165,585.00
	82	Orange	4.000	Concrete	Oct. 27	228.000.00	79,800.00
	83	Suffolk	22.500	do	do	1,061,500.00	449,525.00
	85	Rensselaer and Washington	6.200	do	Oct. 25	353,400.00	123,690.00
Ohio	178	Medina	2.282	do	Oct. 13	92,400.00	45,600.00
	179	do	1.806	do	do	76,500.00	36,120.00
Oklahoma	42	Osage	5.900	do	Oct. 11	233,049.99	116,524.99
Oregon	21	Clatsop	[1] 14.800	Asphalt, concrete on macadam base		[1] 318,835.00	[1] 159,417.50
	49	Clackamas	5.700	Concrete	Oct. 27	242,783.75	114.000.00
South Carolina	65	Newberry	4.231	Top soil	Oct. 16	40,356.10	20,000.00
	90	Fairfield	5.439	do	Oct. 23	40,217.83	20,108.91
Texas	25	Dallas	30.000	Macadam, asphalt surface	Oct. 9	[1] 19,999.98	[1] 9,999.99
	50	Coke	18.650	Sand-clay and gravel	Oct. 28	[1] 44,836.00	[1] 20,000.00
	207	Chambers	11.350	Shell	Oct. 11	129,708.74	38,000.00
	214	Cooke	9.500	Gravel	Oct. 23	169,619.56	80,000.00
	215	Wood	4.570	do	Oct. 21	72,696.16	35,000.00
Vermont	21	Franklin	2.300	Macadam	Oct. 23	103,127.75	51,563.87
West Virginia	103	Tyler	4.160	Earth	Oct. 21	46,867.40	23,433.70
Wisconsin	121	Bayfield	16.800	do	do	170,820.50	57,000.00
	169	Sheboygan	5.150	Concrete	Oct. 13	222,084.81	75,000.00
Wyoming	16	Lincoln	1.929	Selected material	Oct. 11	24,722.50	12,361.25
	103	Park		Bridge	Oct. 23	7,282.00	3,641.00
	106	Washakie	2.746	Selected material	Oct. 21	33,000.00	16,500.00

[1] Withdrawn or canceled.
[2] Revised statement. Amounts given are decreases from those in the original statement.
[3] Revised statement. Amounts given are increases over those in the original statement.

PROJECT AGREEMENTS EXECUTED IN OCTOBER, 1920.

State.	Project No.	County.	Length in miles.	Type of construction.	Project agreement signed.	Estimated cost.	Federal aid.
Alabama	14	Talladega		Gravel	Oct. 11	[1] $13,866.25	[1] $6,933.12
	17	Greene		Clay or gravel	...do...	[2] 24,734.10	[2] 12,367.05
	51	Russell		Sand-clay	...do...	[2] 17,114.30	[2] 8,557.15
	53	Coosa		Earth	...do...	[2] 879.96	[2] 439.98
	54	.....do		.....do	Oct. 13	[2] 460.88	[2] 230.44
Arizona	10	Maricopa		Steel or concrete	Oct. 19	[2] 5,306.84	[2] 2,653.42
	15C	Graham		Bridge	Oct. 19	43,105.37	21,552.68
	18	Cochise		Earth	Oct. 21	[2] 11,497.67	[2] 5,748.83
	19B	Yavapai	3.976	Gravel	...do...	83,839.85	41,919.92
	23D	Pinal	8.664	.....do	...do...	156,114.53	78,057.26
	24	Coconino	15.114	Volcanic	Oct. 21	202,588.37	101,294.18
	26	Yuma	18.630	Gravel	Oct. 19	237,860.04	118,930.02
Arkansas	8	Lonoke	[3] 1.970	Waterbound macadam	Oct. 23	[3] 8,871.72	[3] 3,169.00
	35	Pulaski	8.810	Macadam and asphalt	Oct. 19	188,419.60	87,500.00
	37	Washington	14.500	Gravel	Oct. 11	32,574.74	15,000.00
	96	Poinsett	4.250	Concrete	...do...	148,104.00	42,000.00
Colorado	38	Morgan	2.727	.....do	Oct. 25	102,715.63	51,357.81
	40	Yuma	3.504	Gravel	...do...	29,890.85	14,945.42
	52	Teller	4.884	Earth	...do...	23,628.48	11,814.24
	58A	Powers	3.129	Gravel	...do...	44,978.80	22,489.40
	59	Bent	9.921	.....do	...do...	85,092.02	42,546.01
	74	Moffat	6.493	.....do	...do...	74,953.59	37,476.79
	75	Grand	6.847	Earth	...do...	24,820.56	12,410.28
	80	Routt	6.647	.....do	Oct. 27	63,715.19	31,857.59
	86	Larimer	1.347	Concrete	...do...	57,686.20	26,940.00
	91A	Las Animas		Gravel	...do...	[2] 13,317.14	[2] 6,658.57
	106	Routt	1.357	.....do	...do...	27,980.70	13,990.35
	107	Moffat	3.970	.....do	...do...	59,026.67	29,513.33
	109	Mesa	.957	Concrete	Oct. 25	48,810.25	19,120.00
	117	El Paso	.929	.....do	Oct. 27	46,522.48	18,885.80
	118	.....do	2.761	.....do	Oct. 25	136,726.37	55,220.00
	128	Rio Blanco	1.799	Shale	...do...	17,342.50	8,671.25
Georgia	52	Evans	13.125	Sand-clay	Oct. 19	72,272.47	30,528.29
	66	McDuffie		Topsoil	Oct. 9	[2] 6,219.94	[2] 3,109.97
	92	Wilkes	5.236	Concrete	...do...	192,403.05	96,201.52
Idaho	23A	Twin Falls	11.269	Bituminous concrete	Oct. 5	553,728.26	225,386.00
	1C,D	Mitchell	6.856	Concrete	Oct. 27	384,332.83	102,840.00
Kansas	22, A to E, F to I	Finney	27.763	Brick or concrete	Oct. 6	1,817,028.37	416,445.00
	26B	Rice	5.022	.....do	...do...	347,745.24	74,490.00
	37C,D,E	Montgomery	8.452	Gravel and concrete	...do...	246,224.50	123,112.25
	41B	Atchison	.761	Concrete	...do...	72,881.20	11,415.00
	53A, B,C,Z	Doniphan	6.103	Concrete, bituminous concrete or bituminons macadam.	...do...	318,828.27	91,545.00
Kentucky	4	Carter		Earth	June 30	[1] 10,534.37	[1] 5,267.19
Louisiana	1	Rapides		Gravel	Oct. 18	[2] 39,571.76	[2] 26,971.22
	23	Lafayette	10.420	.....do	Oct. 14	131,211.33	65,605.66
	30	Allen	10,240	.....do	...do...	119,647.66	35,500.00
Maryland	10	Montgomery	.355	Concrete	...do...	[2] 21,626.00	[2] 7,100.00
	34	Anne Arundel		Gravel	...do...	[2] 66,002.01	[2] 33,001.00
	39	Dorchester	2.350	Concrete	...do...	77,473.71	38,736.85
Massachusetts	17	Essex		Bituminous macadam	Oct. 13	[2] 29,267.54	[2] 4,317.07
	30	Hampden	4.227	.....do	Oct. 19	151,778.00	75,889.00
	34	Worcester	1.359	Concrete	...do...	66,711.15	27,180.00
	36A	Barnstable	10.690	Bituminous concrete	Oct. 13	182,695.70	91,347.85
	40	Worcester	1.011	Concrete	Oct. 19	40,474.50	20,220.00
	41	Middlesex	1.250	Gravel surface treated	...do...	23,034.00	11,517.00
	43	Worcester	.638	Concrete	Oct. 13	63,801.10	12,760.00
Minnesota	28	Grant	5.680	Gravel	Oct. 5	72,349.95	36,174.97
	80	Polk	6.010	.....do	Oct. 30	63,092.00	10,000.00
	79	Hubbard	8.810	.....do	Oct. 5	57,350.04	20,000.00
	91	Kanabec	16.370	.....do	...do...	162,925.92	75,000.00
	94	Polk	9.905	.....do	Oct. 11	61,465.45	10,000.00
	99	Todd	16.940	.....do	Oct. 5	88,700.33	44,000.00
	108	Grant	16.524	.....do	Oct. 25	120,397.68	43,515.10
	109	Redwood	23.320	.....do	Oct. 5	61,178.94	19,000.00
	115	McLeod	7.790	.....do	...do...	66,453.22	15,000.00
	122	Chippewa	12.380	.....do	...do...	98,591.11	15,000.00
	124	Isanti	6.573	.....do	Oct. 23	78,363.93	39,000.00
	126	Rice	3.720	Concrete, asphaltic concrete, or brick	Oct. 5	148,825.67	5,000.00
	128	Chippewa	14.863	Gravel	...do...	140,415.82	20,000.00
	149	Aitkin	9.270	.....do	Oct. 2	50,502.65	20,000.00
	155	Polk	13.490	.....do	Oct. 11	84,788.06	11,000.00
	157	Itasca	.807	Concrete	Oct. 5	44,595.05	5,000.00
	167	Hubbard	9.020	Gravel	...do...	71,648.31	5,000.00
	169	.....do	7.430	.....do	Oct. 25	44,975.01	15,000.00
	174	Dodge	3.010	.....do	Oct. 2	49,525.80	8,462.22
	185	Jackson	6.010	.....do	Oct. 25	55,271.02	20,000.00
Montana	2	Wibaux	8.773	.....do	Oct. 26	48,761.91	24,380.95
	4	Carbon	[2] 2.624	.....do	Oct. 16	[2] 66,139.21	[2] 33,069.61
	9	Madison		Selected material	Oct. 29	[2] 9,811.88	[2] 4,905.94
	12	Broadwater and Gallatin	3.310	Earth	Oct. 26	45,386.62	22,693.31
	28	Fergus	7.890	.....do	Oct. 11	68,684.64	34,342.32
	36	Powell	4.120	Gravel	Oct. 26	29,155.48	14,577.74
	53B	Yellowstone	11.016	.....do	...do...	105,352.57	52,676.28
	61	Wibaux		.....do	Oct. 9	[2] 8,934.72	[2] 4,467.36
	65AB	Teton and Cascade	35.850	.....do	Oct. 16	316,090.20	158,045.10
	66	Blaine	12.660	.....do	...do...	162,688.47	81,344.23
	77	Lewis and Clark	9.057	Gravel and concrete	Oct. 9	128,737.27	59,204.32
	103C	Fergus	21.760	Concrete	Oct. 16	131,345.56	65,672.78
	115	Wheatland	9.156	Gravel	Oct. 9	74,364.90	37,182.45
	1	.....do	6.250	Earth and gravel	...do...	62,193.08	31,096.54
New Hampshire	15	Merrimack		Gravel	Oct. 15	[2] 5,304.15	[2] 2,652.08
	33	Rockingham		Surface-treated gravel	...do...	[2] 966.06	[2] 483.03
	47	Sullivan, Merrimack		Gravel	...do...	[2] 1,254.00	[2] 627.00
	41	Merrimack	1.110	.....do	...do...	20,052.36	10,026.18
	45	Strafford	.750	.....do	Oct. 14	10,030.79	5,015.39
	52	Sullivan, Merrimack		.....do	Oct. 15	[1] 655.22	[1] 327.61
	55	Strafford	.550	.....do	...do...	9,905.61	4,952.80

[1] Modified agreement. Amounts given are increases. Second revision.
[2] Modified agreement. Amounts given are increases over those in the original agreements.
[3] Canceled.

PROJECT AGREEMENTS EXECUTED IN OCTOBER, 1920—Continued.

State.	Project No.	County.	Length in miles.	Type of construction.	Project agreement signed.	Estimated cost.	Federal aid.
New Hampshire (continued)	92	Merrimack	0.500	Gravel	Oct. 9	$8,013.11	$4,006.55
	94	Cheshire	.390	do	Oct. 14	12,055.08	6,027.54
	95	do	.790	Gravel and bituminous macadam	Oct. 15	14,879.30	7,439.65
	96	Hillsborough	.410	Gravel	do	7,489.15	3,744.57
	100	Belknap	.767	do	Oct. 14	16,094.54	8,047.27
	102	Merrimack	.910	do	Oct. 13	10,095.36	5,047.68
	103	do	.340	do	Oct. 14	10,003.76	5,001.88
	104	do	.660	do	Oct. 13	9,994.93	4,997.46
	105	do	1.140	do	Oct. 9	14,097.73	7,048.86
	108	Grafton	.380	Bituminous macadam	Oct. 15		[4] 1,406.96
	114	Strafford	1.420	Gravel	Oct. 14	24,887.06	12,443.53
	115	Rockingham	1.210	Gravel, surface-treated	Oct. 13	15,039.97	7,519.98
	119	Cheshire and Hillsborough	.430	Bituminous macadam	Oct. 14	14,835.48	7,417.74
	120A	Carroll	2.590	Gravel	Oct. 15	32,645.03	16,322.51
	122	Cheshire		Bridge	Oct. 14	16,388.90	8,194.45
	123	Hillsborough	.300	Gravel	do	7,992.05	3,996.02
	124	Belknap	1.240	Bituminous macadam	Oct. 15	17,957.44	8,978.72
	129	Rockingham	1.630	Mod. asphalt	Oct. 9	28,487.25	14,243.62
	97	Cheshire	.530	Bituminous macadam	do	19,718.71	9,859.35
New Mexico	4	Valencia		Earth	Oct. 11	[5] 5,408.55	[5] 2,704.28
	12	Chaves		Crushed rock or gravel	do	[5] 19,467.69	[5] 9,733.84
	13	Valencia		Macadam	do	[2] 8,614.86	[2] 4,307.43
	24	Roosevelt		Caliche	do	[2] 18,581.83	[2] 9,290.92
	31	Sierra	3.902	Gravel	do	23,360.14	11,680.07
	33	Moro		do	do	[2] 2,906.27	[2] 1,453.14
	47	Grant		do	do	[2] 15,215.03	[2] 7,607.52
North Carolina	48A	Northampton		Sand-clay	Oct. 30	[2] 3,959.51	[2] 1,979.75
Rhode Island	2	Washington		Bituminous macadam	Oct. 28	[1] 8,800.00	[1] 4,400.00
	8	Kent	4.400	Concrete	do	232,245.75	88,000.00
	9	Washington	2.290	do	do	117,854.19	45,800.00
	10	Newport	2.030	Bituminous macadam	Oct. 26	114,111.36	40,600.00
South Carolina	6	Marion		Bridges	Oct. 23	[1] 8,349.57	[1] 4,174.78
	17	Calhoun		do	Oct. 27	[1] 1,924.38	[1] 926.19
	21	Edgefield		Topsoil	do	[2] 10,789.47	[2] 5,394.74
	25	Lexington		Sand-clay	do	[2] 70,212.97	[2] 16,019.94
	30	Pickens		Topsoil	do	[2] 7,279.00	[2] 3,639.50
	46	Kershaw		Gravel	do	[2] 489.54	[2] 244.77
	50	Dillon	19.767	Sand-clay	do	131,604.38	40,714.39
	71	York	6.515	Topsoil	do	34,899.96	14,000.00
	73	Spartanburg	12.542	Topsoil and asphaltic concrete	do	124,843.98	18,150.82
	80	Williamsburg	11.027	Sand-clay	do	164,721.08	78,588.24
	92	Union	5.327	Topsoil	do	31,486.25	7,414.19
	100	do	3.729	do	do	36,588.18	14,828.38
	104	Calhoun	.669	Asphaltic concrete	do	49,948.50	13,380.00
South Dakota	25	Brookings	11.800	Gravel	Oct. 22	132,659.56	66,329.78
	26	Kingsbury		do	do	[2] 9,653.45	[2] 4,826.73
	29	Corson	9.053	Earth and gravel	do	80,321.63	40,160.81
	31	Beadle	8.492	Gravel	do	99,394.88	49,697.44
	37	Potter and Faulk	12.940	Earth	do	72,326.21	36,163.10
	41	Marshall and Roberts	24.460	Earth and gravel	do	248,766.07	124,383.03
	55	Deuel	9.605	Gravel	do	118,447.56	59,223.78
	20A	Walworth	8.984	Earth	Oct. 21	81,387.58	40,693.79
Texas	4	Travis		Gravel, surface treated	Oct. 9	[1] 15,681.60	[1] 7,128.00
	25	Dallas	3.000	Macadam, asphalt surface	do	[3] 25,155.87	[3] 10,000.00
	26	Titus		Gravel, bituminous surface	Oct. 4	[2] 22,762.51	[2] 20,048.70
	36	Caldwell		Gravel	do		[2] 25,000.00
	46A	Gillespie	17.631	do	Oct. 9	169,614.85	68,680.00
	48	Bosque	51.970	do	do	157,613.21	59,900.81
	53A	Jefferson	2.500	Concrete, with asphalt top	Oct. 10	115,320.41	43,000.02
	56A	do		Asphalt	Oct. 4	88,455.46	44,227.72
	56B	do	3.010	Concrete	do	[2] 19,689.85	[2] 7,687.40
	78	Freestone		Gravel and sand-clay	Oct. 20	[2] 17,875.97	[2] 8,937.98
	83	Brown	10.750	Gravel, stone, macadam, earth	Oct. 4	90,867.43	22,500.00
	85	Freestone		Waterbound macadam	Oct. 9	[2] 15,032.07	[2] 18,000.00
	87	Travis	10.000	Bituminous	do	35,816.00	16,426.50
	105	Wood		Iron ore gravel	do	[1] 19,079.41	[1] 8,750.00
	108	Kendall	5.778	Gravel, surface treated	do	48,102.90	24,051.45
	113 1 and 2	Hill		Gravel, bituminous top	Oct. 4	[4] 24,128.02	[2] 25,000.00
	125	Liberty	.332	Gravel	Oct. 9	[4] 6,382.55	[2] 2,410.65
	142	Harrison	23.243	Gravel, surface treated	do	431,064.39	200,000.00
	146	Delta	11.310	Gravel	Oct. 4	323,800.77	120,000.00
	147	Smith	24.560	Gravel and gravel, bituminous top	Oct. 9	380,769.91	190,384.95
	153	Rains	22.340	Gravel	do	269,088.99	100,000.00
	157	Galveston	1.560	Rock asphalt	do	101,369.84	30,000.00
	159	Shelby	31.512	Gravel	do	480,427.73	169,614.58
	164	Erath	31.267	Rock and clay, gravel	do	296,948.58	148,474.29
	170	Fort Bend	13.010	Gravel, surface treated	do	250,125.74	80,000.00
	174	Randall	17.030	Earth	do	28,708.22	14,000.00
Washington	42	Grays Harbor		Concrete	Sept. 20	[2] 5,000.00	[2] 2,500.00
	56	Walla Walla	3.410	do	Oct. 25	158,694.60	72,913.50
	63	Stevens	6.460	Gravel	Oct. 14	54,825.04	27,000.00
West Virginia	23	Kanawha		Asphalt, concrete	Oct. 12	[2] 23,052.00	[2] 14,280.00
	45	Putnam		Concrete, brick, or bituminous macadam	do	[2] 15,931.12	[2] 22,541.71
Wisconsin	29	Washburn and Barron		Grading, draining, and culverts	Oct. 30	[2] 7,865.78	[2] 2,621.93
Wyoming	12	Lincoln	.251	Gravel	Oct. 4	[2] 22,417.45	[2] 11,208.72
	31	Natrona		Warrenite	Oct. 6	[2] 1,431.71	[2] 715.86
	41	Crook	10.089	Selected material	do	80,245.59	40,122.79
	42	Sheridan	9.355	do	Oct. 11	106,001.28	53,000.64
	46	Laramie	25.193	do	Oct. 4	109,733.82	54,866.91
	54	Big Horn	.639	do	do	[1] 7,839.00	[1] 3,919.50
	57	Converse	4.251	do	do	98,764.06	49,382.03
	66B	Campbell	8.958	do	do	99,262.08	49,631.04
	71	Washakie	3.131	do	do	32,241.66	16,120.83
	76	Park	4.964	do	do	52,948.83	26,474.41
	83	Sheridan	14.129	do	Oct. 8	208,177.37	104,088.68

[1] Modified agreement. Amount given are increases. Second revision.
[2] Modified agreement. Amounts given are increases over those in the original agreement.
[3] Canceled.
[4] Modified agreement. Amounts given are decreases from those in the original agreements.
[5] Modified agreement. Increase. Third revision.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS.

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

REPORTS.

*Report of the Director of the Office of Public Roads for 1916. 5c.
*Report of the Director of the Office of Public Roads for 1917. 5c.
Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.

DEPARTMENT BULLETINS.

Dept. Bul. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
136. Highway Bonds.
220. Road Models.
230. Oil Mixed Portland Cement Concrete.
249. Portland Cement Concrete Pavements for Country Roads.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
314. Methods for the Examination of Bituminous Road Materials.
347. Methods for the Determination of the Physical Properties of Road-Building Rock.
*348. Relation of Mineral Composition and Rock Structure to the Physical Properties of Road Materials. 10c.
370. The Results of Physical Tests of Road-Building Rock.
373. Brick Roads.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
389. Public Road Mileage and Revenues in the Central, Mountain, and Pacific States, 1914.
390. Public Road Mileage in the United States, 1914. A summary.
393. Economic Surveys of County Highway Improvement.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
414. Convict Labor for Road Work.
463. Earth, Sand-Clay, and Gravel Roads.
532. The Expansion and Contraction of Concrete and Concrete Roads.
537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests.
555 Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials.
583. Reports on Experimental Convict Road Camp, Fulton County, Ga.
586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916.
660. Highway Cost Keeping
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
691. Typical Specifications for Bituminous Road Materials.
704. Typical Specifications for Nonbituminous Road Materials.
724. Drainage Methods and Foundations for County Roads.
Public Roads, Vol. I, No. 11. Tests of Road-Building Rock in 1918.

OFFICE OF PUBLIC ROADS BULLETINS.

Bul. *37. Examination and Classification of Rocks for Road Building, Including Physical Properties of Rocks with Reference to Their Mineral Composition and Structure (1911.) 15c.
*43. Highway Bridges and Culverts. (1912.) 15c.
*45. Data for Use in Designing Culverts and Short-Span Bridges. (1913.) 15c.

* Department supply exhausted.

OFFICE OF PUBLIC ROADS CIRCULARS.

Cir. 89. Progress Report of Experiments with Dust Preventatives, 1907.
*90. Progress Report of Experiments in Dust Prevention, Road Preservation, and Road Construction, 1908. 5c.
*92. Progress Report of Experiments in Dust Prevention and Road Preservation, 1909. 5c.
*94. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1910. 5c.
98. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1911.
*99. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1912. 5c.
*100. Typical Specifications for Fabrication and Erection of Steel Highway Bridges. (1913.) 5c.

OFFICE OF THE SECRETARY CIRCULARS.

Sec. Cir. 49. Motor Vehicle Registrations and Revenues, 1914.
52. State Highway Mileage and Expenditures to January 1, 1915.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
65. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Aid Road Act.
72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
74. State Highway Mileage and Expenditures for the Calendar Year 1916.
77. Experimental Roads in the Vicinity of Washington, D. C.
Public Roads Vol. I, No. 1. Automobile Registrations, Licenses, and Revenues in the United States. 1917.
Vol. I, No. 3. State Highway Mileage and Expenditures in the United States, 1917.
Vol. I, No. 11. Automobile Registrations, Licenses, and Revenues in the United States, 1918.

DEPARTMENT CIRCULAR.

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS.

F. B. 338. Macadam Roads.
*505. Benefits of Improved Roads. 5c.
597. The Road Drag.

SEPARATE REPRINTS FROM THE YEARBOOK.

Y. B. Sep. *638. State Management of Public Roads; Its Development and Trend. 5c.
727. Design of Public Roads.
739. Federal Aid to Highways, 1917.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH.

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cement.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced Concrete Slabs Under Concentrated Loading.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.
Vol. 17, No. 4, D-16. Ultra-Microscopic Examination of Disperse Colloids Present in Bituminous Road Materials.

* Department supply exhausted.