REPORT OF COMMITTEE ON HIGHWAY DESIGN

A T GOLDBECK, *Chairman*

DESIGN OF JOINTS IN CONCRETE PAVEMENTS

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SYNOPSIS

This paper presents a general discussion of joints in concrete pavements Current design practice, experience, and certain investigational data are briefly reviewed The several fundamental types of joints are described and discussed with special reference to basic structural requirements Special emphasis is laid upon the rational design of joint details and certain analytical procedures are developed for the rational detailing of edge bars, slip dowels, tie bars and other features essential to an adequate functioning of joint edges Application of theoretical principles of design, with respect to basic structural requirements, emphasizes apparent widespread inconsistencies of current practice as reflected by the widely varying joint details prescribed by the various states The slip dowel, widely used as a means of strengthening joint edges and apparently used generally on some arbitrary basis of selection as to size, length and spacing, is analyzed and discussed at length Theoretical analysis thus indicates that many slip-dowel designs, now in common use, must necessarily be decidedly inefficient in serving the structural function which they are intended to perform

TYPES AND PURPOSES OF JOINTS

Joints as Related to Pavement Design The external conditions to which concrete pavements are subjected are numerous and of varying character Some of these stress-causes induce stresses of a distinctly localized character and are not greatly affected by the lateral dimensions of the slab, when those dimensions are of the order commonly found in pavements Others, however, are a direct function of slab length or width and accordingly their evaluation is dependent upon the existence of slab units having definite lateral dimensions Predetermined slab size is thus essential to any rational analysis of the stresses occurring in concrete pavements, especially those stresses induced by subgrade friction, since obviously no basis of stress computation can be set up for this type of external force unless the magnitude of that force, with respect to any given section of resistance, be fixed by definite slab length

and width Joints of some kind are therefore necessary in order to establish definite slab units of predetermined size

It is evident that each slab unit must in itself be structurally adequate to resist the external forces and conditions to which it may be subjected, and that each unit must also function structurally with its neighbor in such a way as to maintain the strength and utility of the pavement as a whole The structural design of a concrete pavement thus becomes a problem of attaining satisfactory crack control within each individual slab unit, and also the provision of joints of suitable and adequate character to maintain the necessary structural relationship between adjoining slabs

Types of Joints Structurally, there are but two basic types of joints used in concrete pavements, (a) the closed or "hinged" type, and (b) the open or "free" type Joints formed by cracks may be included in either one of these two structural groups, depending upon whether or not distributed reinforcement has been provided in the original design to maintain cracks as closed joints, otherwise they will become open or free joints when adjoining slab sections contract

Closed or Hinged Type Closed joints are utilized to relieve the slab of bending stresses but at the same time they are intended to be capable of resisting shear, a structural action commonly referred to as "transfer of load " As commonly built, joints of this type are no doubt capable of providing some resistance to bending moment, due to the fact that the tie steel used to keep the joint closed, although usually placed at the center of the slab depth, will tend to act in conjunction with some compression contact either above or below the tie steel, thus giving the section some degree of resistance to bending However, experience has shown that these joints as commonly built, utilized principally as center longitudinal joints, possess sufficient flexibility with respect to bending to give the slab all the relief needed at such sections

The closed or hinged joint, therefore, is structurally interpreted as being one which is incapable of resisting bending moment but capable of resisting transverse shear The shear-resisting feature of such joints is obtained basically by giving the joint a face interlock of some kind This interlock may be provided in several ways by a preformed interlocking ridge and depression formed by the installation of the familiar and widely used deformed metal plate, or by the use of wooden strips attached to the side forms, producing a longitudinal indentation or groove in one slab into which the concrete flows when the adjoining slab is subsequently poured, thus forming the necessary tongue and groove arrangement

The principle of tongue-and-groove interlocking is also obtained by use of the so-called "dummy" or cracked joint This joint is essentially no different in structural principle from the preshaped tongue-and-groove joint as provided by the deformed metal plate The

difference lies merely in the fact that the dummy joint, as formed by subsequent cracking of the lower part of the slab, provides two rough jagged faces which are capable of interlocking themselves and are accordingly shear resistant by virtue of an innumerable number of miniature tongues and grooves, so to speak

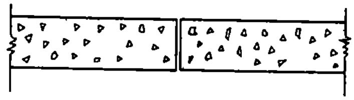
Since the basic structural function of the closed joint, regardless of its specific form, is to be shear resistant, it is necessary that the faces of such joints be prevented from separating in order to assure at all times a close interlock of the tongue and groove formation Tie steel of some kind must necessarily be provided for this purpose Such steel is commonly utilized in the form of relatively short individual tie bars bonded in both adjoining slabs and thus acting in tension to resist any forces tending to produce lateral separation of the joint faces These bars are sometimes referred to as dowel bars It seems desirable, however, to reserve the term "dowell" to apply to another kind of short individual bars crossing joints and which serve an entirely different structural function The bars crossing a hinged joint, therefore, should be designated as "tie bars" and not "dowels" since they are not intended to perform any doweling function but are intended to serve merely to hold the faces of the joint in intimate contact, the tongue and groove interlock, and not the tie bars, resisting the shear on the joint

Open or Free Type. The open or free type joint is one wherein provision is made for the free opening and closing of the joint under lateral movement of adjoining slabs This type is used for expansion, contraction, and construction joints Obviously, such joints are incapable of resisting bending moment but they may or may not be shear resistant When it is desired to render free joints shear resistant, use is made of slip-dowels, which are relatively short bars extending across the joint, the dowel being inserted in a bondless socket in one of the slabs, thereby permitting free opening or closing of the joint as adjoining slabs shorten or lengthen due to changes in temperature and moisture content From a structural standpoint, both contraction and expansion joints are free type and must be designed on the hypothesis that joint faces are actually separated The principles of edge strengthening along contraction joints and expansion joints are essentially the same, the only difference being in the width of gap to be provided for in either case

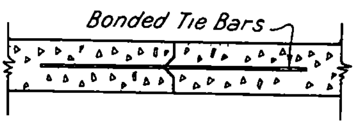
Cracks A crack is usually a joint of the most undesirable type since its alignment and character of formation can not be controlled. Since cracks are joints that only potentially exist, protection of pavement strength, with respect to crack joints, can be accomplished only by anticipating crack occurrence and making adequate provision for joint strengthening on the hypothesis that a crack may eventually occur at the slab's most critical section.

In view of the strong probability of cracks occurring in any slab unit, as a result of the variable and uncertain conditions to which pavements

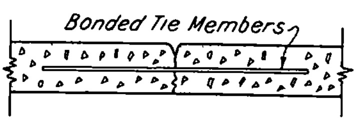
are subjected, it is frequently considered advisable to utilize steel reinforcement of such type and amount as will provide closely-spaced units, distributed throughout the entire area of each individual slab unit, to serve as tie members across any crack that may subsequently form a joint in the pavement, regardless of what location such a joint may have or what direction it may take As long as the faces of cracked joints can be held closely together, thereby maintaining secure face interlock, the high frictional resistance of such faces will assure simultaneous rather than independent action of the two crack edges and thus render such joints structurally as adequate as was the original pavement section before the crack appeared



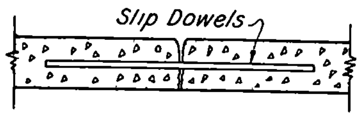
SMOOTH-FACE BUTT JOINT
Figure 1



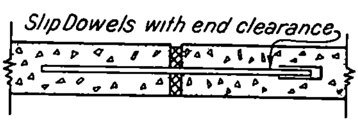
TONGUE AND GROOVE TIED JOINT
Figure 2



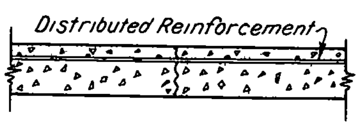
DUMMY TYPE TIED JOINT
Figure 3



DUMMY TYPE DOWELED JOINT
Figure 4



DOWELED EXPANSION JOINT
Figure 5



TIED CRACK JOINT
Figure 6

Figures 1 to 6 Types of Pavement Joints

Longitudinal Joints Formerly, when 16- and 18-foot pavements were constructed without any longitudinal joint, longitudinal cracking was very prevalent The well-known Bates Road Test developed the idea of building pavements in comparatively narrow widths with the result that the use of longitudinal joints of some kind is now practically universal Experience has definitely shown that where longitudinal joints are provided so as to divide the pavement into widths or lanes of approximately 10 feet, longitudinal cracking is practically eliminated

Longitudinal joints are usually closed joints of the tongue-and-groove or dummy type held together by tie steel in cases where the slab is a two-lane pavement In the case of four-lane pavements, common

practice consists in constructing the pavement as two double lane units, the longitudinal joint dividing the two units usually being of the butt type, the edges of which are designed to act as independent free pavement edges Butt type longitudinal joints are also used sometimes even in two-lane pavements when the pavement is constructed one lane at a time This is a common procedure in localities where fairly heavy traffic must be maintained during construction or in cases where it is desirable to keep construction equipment off the sub-grade while the slab is being poured

Transverse Joints Transverse joints are of three kinds, expansion, contraction, and construction In the case of pavements having a definite predetermined arrangement of transverse joints at reasonably close intervals, the necessity for construction joints seldom arises, except in cases of distinct emergency, normal stopping of the work being timed to conform with the established slab length

The purpose of transverse joints is to establish a definite slab length as a means of eliminating otherwise excessive stresses due to longitudinal movement of the pavement resulting from expansion and contraction Two types of joints are used for this purpose, expansion joints and contraction joints Various materials are utilized for expansion joint fillers, such as poured or premolded bitumen, rubber, cork, and other compressible compounds A new type of joint filler is being rapidly developed, known as the air cushion joint This type of filler consists essentially of a light weight metal sheet formed in a U-shape, the top of the U being closed by a V-shaped piece of metal which permits opening and closing of the joint and which serves also as a reservoir to receive a poured bituminous filler at the pavement surface

Transverse contraction joints are utilized for providing a free joint only when adjoining slabs contract The contraction joint, therefore, is constructed as a closed joint, becoming an open or free joint only when contraction occurs Such joints are generally of the dummy or cracked type, formed by merely cutting in the pavement surface a shallow groove extending to a depth of approximately one-third the thickness of the slab with surface edges rounded to prevent spalling and having a width of about $\frac{1}{2}$-inch to provide the necessary reservoir for bituminous filler

Transverse joints may thus consist either entirely of expansion joints or an arrangement of alternate expansion and contraction joints It is generally considered necessary to provide a total expansion space of at least 1 inch per 100 lineal feet of pavement and, since the contraction joint is fundamentally cheaper than the expansion joint, a common arrangement is to provide expansion joints of suitable width at comparatively wide intervals and provide one or more intermediate contraction joints to fix slab length and safeguard crack control Expansion joints, however, should not be spaced at intervals such as to require joint widths

exceeding 1 to $1\frac{1}{2}$ inches, owing to the difficulty of keeping wide joints properly filled when maximum opening occurs and the objectionable feature of exuding of the filler material when minimum opening occurs Wide joints are furthermore objectionable in case it is desired to dowel the joint, since it is not economically practical to adequately dowel a joint much wider than 1 inch, owing to the high bending and bearing stresses developed in the dowel The spacing of transverse joints fixes the free length of slab and any arrangement adopted has an important influence upon the problem of crack control with respect to each slab unit

Slab Length The proper spacing of transverse joints is largely a matter of judgment based upon experience The ultimate objective, is to accomplish satisfactory crack control, and proper joint spacing therefore involves many factors, such as character of aggregate, curing, subgrade conditions, reinforcement, and other elements influencing the normal uncracked length of slab In some localities conditions are apparently such as to permit a much wider spacing of transverse joints than in others, as revealed by extensive surveys of actual pavement condition Where transverse joints are practically eliminated, by being used only at very wide spacing, experience has shown that slab lengths, as developed by cracking, will be extremely erratic This, together with the fact that slab lengths as established by cracking develop joints of the most undesirable character, would seem to emphasize the importance of definitely fixing a reasonably short slab length in order to safeguard a more dependable behavior of the pavement in service Current practice in this regard indicates a distinct preference for slab lengths of from 30 to 60 feet

CURRENT DESIGN PRACTICE

State Highway Standards Current practice with respect to the use of joints as well as their details of design is indicated by a recent survey of 1932 standards prescribed by the various State Highway Departments This survey reveals a most striking lack of agreement on the part of highway designers with respect to joints, not only in regard to design details of the joint itself but even as to the basic principle of whether or not predetermined joints are required in concrete pavements However, those states using practically no transverse joints whatever are decidedly in the minority. In view of the wide variation in joint details, it is quite evident that design of the pavement joint itself is at the present time based largely upon the use of details arbitrarily chosen without consistent regard to the basic structural principles involved

Longitudinal Joints The most outstanding similarity in practice is the almost universal use of some type of longitudinal joint to divide pavements into lanes of 9- or 10-foot widths Such joints are used by all except three states in all concrete pavements.

The three general types of longitudinal joint are the butt, the deformed metal plate, and the surface groove or dummy. The butt type is used exclusively by 23 per cent of the states, the metal plate exclusively by 25 per cent, and the groove by 9 per cent The use of the butt type is limited, of course, to lane-at-a-time paving

Tie bars are used generally through longitudinal joints, only 23 per cent of the states not using them. Generally those states using no tie bars employ the uniform thickness cross section, use butt joints or interior thickened edge Although various sizes and spacings of tie bars are employed, the common arrangement is to use ½-inch round bars, 4 feet long and spaced at 5-foot intervals

Transverse Joints Transverse joints are used by all except four State Highway Departments The spacing of these joints is quite variable, slabs being constructed in lengths of from 15 feet to as high as 1000 feet. The most frequently used spacing is about 40 feet, this length being used in 31 per cent of the designs which employ joints The range of from 30 to 60 feet includes 73 per cent of the joint designs. Transverse joints are more frequently of the expansion type, although a few states use the contraction type exclusively, and 36 per cent use both the expansion and contraction types at alternate intervals in the slab

Expansion joints are generally of the bituminous type, the premolded being specified by 41 per cent of the states using expansion joints, the poured being required by 10 per cent, and either of the two types being required by 43 per cent Two states permit the use of rubber, cork and other compounds and several types of metal joint are being used to a limited extent

Transverse contraction joints are almost always some form of surface groove or so-called "dummy" type, but two states permit the use of the deformed metal plate transversely

Dowels Slip-dowel bars are used through transverse expansion joints in 64 per cent of the expansion joint designs. The size of the dowel and its spacing in practice are extremely variable, practice varying from the use of a ½-inch round bar at 36 inches to a ¾-inch round bar at 10 inches The most prevalent arrangement appears to be the use of ¾-inch round bars at about 28-inch spacing

In transverse contraction joint practice, it is significant to note that only 26 per cent of the designs require the use of any dowel bars whatever In view of the fact that such joints are subject to opening under contraction of the slabs, the omission of these dowels is not consistent with expansion joint practice A few of these non-dowel using designs, require that marginal bars be extended across the joint for a foot or two and rendered bondless for this length

Joint and Corner Bars Extra bars adjacent and parallel to transverse joints are used by seven states Hairpin bent bars, or straight bars

placed to bisect the corner angle at joints, are used by seven states Such corner and slab-end bars are sometimes used at all transverse joints, and in other cases only at expansion joints Several states strengthen joint corners and slab ends through the use of extra sheets or mats of reinforcing steel

LOADS AND IMPACT

Traffic Loads From the standpoint of joint design, maximum wheel loads are of primary importance The total weight of vehicular unit is not necessarily important since strength requirements with respect to the joint are limited by the magnitude of wheel concentration regardless of the disposition of the total load on several axles The simultaneous application of several wheel loads must, however, be considered as in the case of two vehicles passing at a joint and it may thus be necessary to consider several wheel loads although each would be of the maximum amount The magnitude of wheel load to be utilized in joint design depends upon the maximum load for which the particular pavement is designed This varies, but in most states 8,000 pounds is established as a legal maximum load limit and may accordingly be considered as the maximum load most generally used

Impact The impact effect of moving loads is not generally considered of great importance in the design of the concrete pavement slab particularly when such loads are applied through pneumatic tires and in view of the rigid evenness requirements now prescribed for practically all concrete pavements However, at joints it would seem advisable to give some consideration to impact effect The pavement surface at joints may not always have a common elevation, also the raising of compressible joint filler may create the condition of a slight height obstruction, thereby creating impact effect under moving loads Investigations have indicated that the effect of impact may be very material, depending upon the height and type of obstruction Tests reported by Buchanan and Reid (1) indicate that even with pneumatic tires the impact effect due to an inclined obstruction 1 inch high may be as much as 150 per cent of the static load

Results of investigations recently conducted by the Illinois Department of Highways (2) are significant with especial reference to impact effects on joints One conclusion of this investigation states

"More danger from construction joints and transverse cracks than from surface variations Sudden loads, which occur when vehicles pass from one slab to another across a free joint were seen to have a maximum effect equal to static loads 1 5 times as great as the wheel loads producing them The Illinois standard pavement is provided with a $\frac{3}{4}$-inch round bar along the edge of the slab, which to some degree relieves the suddenness of the application of loads near a corner The effect of an 8,000-lb (legal limit wheel) load near the center can never

exceed that of a static load of 12,000 lb., which should be the maximum possible effect of an 8,000-lb. wheel load on Illinois pavements."

Investigations would thus seem to indicate that an impact allowance of approximately 50 per cent of the static wheel load should generally be made in designing free-type joints. In the case of closed joints, it would hardly seem necessary to make an impact allowance of this amount. In the case of expansion joints some impact allowance is probably desirable. Apparently 50 per cent should be used. Regarding the Illinois tests where this amount was observed, the joints, however, were free joints without adequate doweling. In case dowels of adequate size and spacing are provided, joint alignment will undoubtedly be better maintained and under such conditions it is very probable that not more than 25 per cent impact allowance would be sufficient.

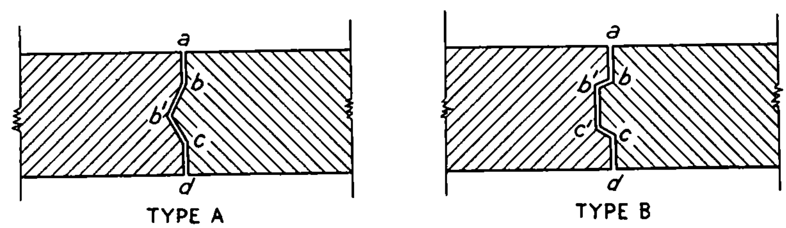


Figure 7 Types of Tongue-and-Groove Formation

NOTATION

In developing certain formulas, herewith presented as applicable to the various features of joint design, the following notation is used:

W = maximum wheel load (lbs.)
P = load-transfer capacity of a single dowel (lbs.)
L = free length or width of slab (feet)
w = average weight of slab (lbs. per sq. ft.)
h_e = exterior-edge thickness of slab (ins.)
h_c = interior thickness of slab (ins.)
l = length of tie bar or dowel bar (ins.).
s = spacing of tie bars or dowel bars (ins.)
z = maximum width of joint (ins.)
d = diameter of bars (ins.)
A_s = cross-sectional area of steel (sq. ins. per ft. width)
f_s = tensile or bending stress in steel (lbs. per sq. in.)
f'_s = shearing stress in steel (lbs. per sq. in.)
S = bending stress in concrete (lbs. per sq. in.)
f_c = bearing stress on concrete (lbs. per sq. in.)
M = bending moment (in.-lbs.)
c = coefficient of subgrade friction

DESIGN OF CLOSED JOINTS

Size and Shape of Tongue and Groove Two shapes of interlock formation are commonly used for the preformed tongue-and-groove joint, indicated as Type A and Type B in Figure 7. Although Type B could be formed with horizontal bearing surfaces, it is considered advisable always to give the surfaces *bb'* and *cc'* a slight slope in order to avoid a possible tendency for the tongue to bind in the groove under the desired hinge action of the joint In either type, the dimensions *ab*, *bc*, and *cd* must be adequate to prevent shearing off either the tongue or the jaws of the groove, also the projection *bb'* must be sufficient to provide adequate bearing Computation of proper values for these dimensions can not be readily made, principally because of the uncertainty as to lineal distribution of concentrated loads applied to the joint. Tongue and groove dimensions are therefore usually based upon certain arbitrary standards which have been found by experience to give satisfactory service The following, although not necessarily recommended, may be taken as typical of satisfactory tongue and groove dimensions now in common use

Typical Tongue-and-Groove Details

(See Figure 7)

	Type A inches	Type B inches
Width of groove (*bc*)	2½ or 3	2½ or 3
Width of groove (*b'c'*)		1½ or 2
Depth of Groove (*bb'*)	1 or 1½	1

Dimensions *ab* and *cd* are approximately equal and vary according to slab thickness

Size of Dummy Joint Groove The purpose of the surface groove, placed in a pavement slab to form a dummy type joint, is two-fold, (a) to weaken the slab section sufficiently to cause it to rupture along the alignment of the groove, and (b) to provide a shallow reservoir for bituminous material to seal the joint Since the shear resistance or load-transfer capacity of a dummy joint of the closed type is provided by interlock of its cracked faces, it is important to conserve as much of the slab thickness as possible for this purpose by cutting the groove no deeper than is absolutely necessary to assure subsequent rupture of the section Experience has shown that a groove depth equal to approximately one-third the slab thickness is sufficient to develop rupture in line with the groove, while a groove width of $\frac{3}{8}$- or $\frac{1}{2}$-inch is ample to hold sufficient filler and to permit satisfactory tooling of the joint edges.

Tie Bars The shear-resisting efficiency of an interlock type of joint is dependent upon keeping the joint tightly closed at all times, hence some means must be provided to prevent the joint faces from separating when the adjoining slabs contract. This is accomplished by the use of

steel tension members extending across the joint and which may consist either of short individual bars or fabricated sheet reinforcement In the preformed type of interlocked joint, where the tongue and groove are shaped by means of wood or metal forms, it is, of course, impossible to tie the joint together by extending a fabricated sheet reinforcement through the joint, and the tie members for such joints must necessarily consist of individual bars which can be merely inserted through small holes provided in the forms Such members are known as "tie bars" and their required diameter, length, and spacing should be computed on a rational basis consistent with the magnitude of force tending to cause the slabs to separate at the joint.

Under the action of any influence tending to cause the slab units of a pavement to contract, any joint will tend to open If this relative movement of slabs at the joint is prevented by tie steel extending across the joint, then the shortening of each slab adjacent to the joint must take place from their free ends toward the joint, rather than from both ends toward their respective centers, and the tie steel must accordingly be adequate to actually drag each slab unit on the subgrade The force of subgrade friction, therefore, constitutes the external force inducing tension in the tie bars Calling $\frac{L}{2}$ the distance from the joint to the nearest free edge of either slab, the maximum force tending to open the joint is equal to $\frac{Lwc}{2}$ pounds per foot length of joint Equating this to the tensile resistance of the tie bars per foot length of joint, gives the familiar formula,

$$A_s = \frac{Lwc}{2f_s} \tag{1}$$

This formula thus enables selection of the required diameter and spacing of bars

The necessary length of tie bar is determined by bond, since the length of embedment in each slab must be sufficient to develop in bond the tensile strength of the bar This required length is readily determined from the bar diameter and the relationship between unit tensile stress and unit bond stress For a given allowable tensile stress the length of bar could be made shorter for deformed bars than for plain bars owing to the higher unit bond permissible with deformed bars However, it is considered good practice to always use the deformed type for tie bars and to make the length of embedment in each slab equal to at least forty diameters The required total length of tie bar may therefore be expressed as,

$$l = 80d \tag{2}$$

Cracks and Reinforcement Structurally any crack in a pavement is a joint, it can not be interpreted otherwise Such joints, can not be

individually "designed" If a crack-joint is to be prevented from creating a section of structural weakness in the slab, its progressive development must be prevented and, in order to accomplish this, the crack must be maintained as a permanently closed joint The only way to introduce this feature is to incorporate in the original design some precautionary means whereby the faces of any subsequently formed crack will be held closely together regardless of when, where or how the crack may occur Consistency in design, with respect to the strengthening of joint edges as well as the basic principle of crack control, would thus require the provision of reinforcing members distributed, both longitudinally and transversely, throughout the entire area of each slab unit

The purpose and justification of such a design procedure is well stated by Clifford Older (3) in a recent comprehensive discussion of crack control in concrete pavements Mr Older states

"To minimize the deleterious effect of cracks that may be caused by conditions not easily evaluated or even anticipated, it is believed that the embedment of a welded mesh having the area of steel in each direction proportioned to slab dimensions and subgrade friction is well worth its cost While embedded steel no doubt promotes other ultimate economies, it is believed that its effect in keeping erratic cracks tightly closed, thus eliminating the evil effect of dirt accumulations and insuring a high degree of doweling action between adjacent section of roughness interlock, is alone sufficient to justify its use"

The basic principle of design with respect to reinforcing steel, introduced for the purpose of protecting joint edges formed by cracks, is the same as that applied above to the design of tie bars By using formula (1) and calling l the distance between free transverse joints, the required sectional area in the longitudinal members of the reinforcement is obtained Similarly, with l equal to the distance between free longitudinal joints, the required sectional area in the transverse members of the reinforcement is obtained

The spacing of members in reinforcement of the distributed type should be such as to provide small units closely spaced rather than large units widely spaced With regard to actual spacing of members the Committee on Reinforced Concrete Pavements of the American Road Builders' Association (4) in its 1932 report states: "It is considered desirable, therefore, to confine the spacing of main longitudinal members as closely to 6 inches as possible, and to restrict the spacing of transverse members to not more than 12 to 16 inches"

EDGE STRENGTHENING AT FREE JOINTS

General Methods In case free joints are not rendered shear resistant, the edges of such joints become free edges of the pavement, exactly the same as the exterior longitudinal edge, and must accordingly have com-

parable strength In case the pavement slab is of uniform thickness throughout, no special provision for joint-edge strengthening is required, but, in case the interior portion of the slab is thinner than the exterior edges, all free joint edges must be strengthened in some way, in order that the load-carrying capacity of the slab may be consistent throughout This strengthening is accomplished by three general methods (a) thickening the slab in the vicinity of the joint similar to that provided along the exterior longitudinal edge, (b) reinforcing the edge with steel in such manner and amount as will render the moment of resistance of the thin section at least equal to that of the exterior longitudinal edge, (c) by the use of steel dowels bridging the open gap of the joint and thus enabling two edges instead of one to carry the applied loads

Edge Thickening The method of strengthening slab edges along free joints by thickening the section is prevalently used for longitudinal joints and has given most satisfactory results, although the thickening of slab ends as a means of strengthening transverse joints has not proved satisfactory and is seldom used Edge thickening at transverse joints introduces not only certain construction objections, such as the necessity for periodic cross-channeling of the subgrade, but has shown a pronounced tendency to promote transverse cracking at or near the section where thickening begins (5) However, in any case where it is desired to strengthen a joint edge by this method, design procedure consists merely in giving the joint edge the same thickness and same rate of reduction as previously designed for the exterior longitudinal edge of the pavement Required edge thickness is thus usually determined by the familiar approximate formula,

$$h_e = \sqrt{\frac{3W}{S}} \tag{3}$$

which is generally considered as being applicable to transverse as well as exterior edges

Edge Reinforcement In the case of thin-center pavements, transverse joints create slab edges which are thinner than the exterior longitudinal edges of the pavement One method of strengthening these edges, without thickening the section or doweling the joint, is to provide edge bar reinforcement The problem of design in this case is to create a reinforced concrete section along the joint edge which will have the same moment of resistance as the plain section along the exterior longitudinal edge Since, under the action of traffic loads, maximum tension occurs in the bottom of the slab, the reinforcement should be located as near the bottom as possible Adequate protection of the steel would require that it be located about 2 inches above the subgrade, hence the depth to the steel would be $(h_c - 2)$ Taking the internal moment arm as equal to nine-tenths the effective depth, the moment of resistance of the reinforced section, per foot width, would be 0 9 $A_s f_s(h_c - 2)$ The mo-

ment of resistance of the plain exterior-edge section, per foot of width, would be $2Sh_e^2$ Equating these, the expression for required sectional area of steel per foot width is,

$$A_s = \frac{2.2\ Sh_e^2}{f_s(h_c - 2)}$$

The question next arises as to what width of section requires this strengthening Westergaard (6) has shown by theoretical analysis that the edge moment caused by traffic loads is what might be termed a "point moment," its value diminishing rapidly, both along and normal to the edge, as the distance from the point of application of the load increases Moment diagrams developed by Westergaard indicate that the positive moment decreases to a value equal to half the maximum in a

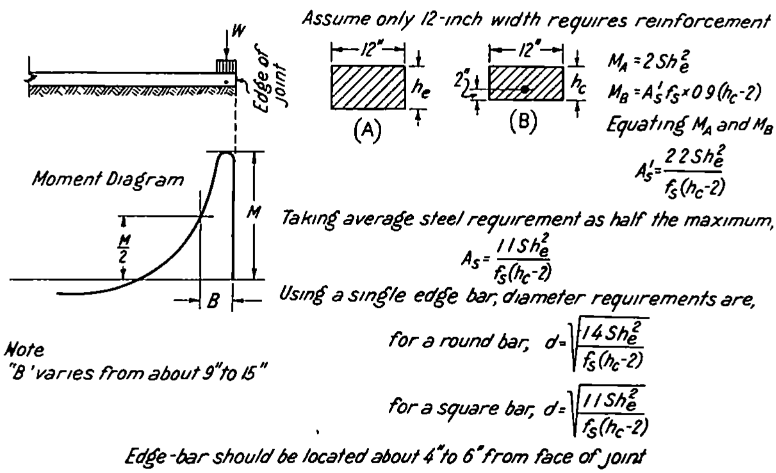


Figure 8. Joint Strengthening with Tensional Edge-Bar

distance of approximately 12 to 15 inches from the joint edge for average slab thickness and subgrade reaction Teller (10) has found experimentally that by moving the load a distance of 12 inches in from the edge of the pavement reduced the edge stress approximately 50 per cent It is thus evident that the section requires strengthening for a width of not more than about 12 inches adjacent to the joint

Another condition to be considered is the fact that, since the need for reinforcement decreases from a maximum requirement of A_s at the edge to zero at a distance of about one foot from the edge, the average steel requirement for the 12-inch width would be only about half the maximum edge requirement It would thus seem permissible to actually use only 50 per cent of A_s as determined above and say that the sectional area of steel, required to reinforce the necessary 12-inch width adjacent to the joint edge, is

$$A_s = \frac{1.1\ Sh_e^2}{f_s(h_c - 2)} \tag{4}$$

Except for unusual conditions, it is generally preferable to utilize this bottom steel as a single marginal bar, located about 4 inches from the joint face, in which case use of formula (4) gives, as the required bar diameter,

for a round bar,

$$d = \sqrt{\frac{1\ 4\ Sh_e^2}{f_s(h_c - 2)}} \tag{5}$$

for a square bar,

$$d = \sqrt{\frac{1\ 1\ Sh_e^2}{f_s(h_c - 2)}} \tag{5a}$$

Corner Strengthening The strengthening of slab corners should be viewed as a feature of joint design, since corners are created only by the presence of joints and the question of their adequate strength may involve the use of dowels or some other form of localized strengthening If the corner is formed by the intersection of a longitudinal edge and the edge of a free undoweled transverse joint, the corner must be of adequate strength to carry the full burden of the maximum wheel

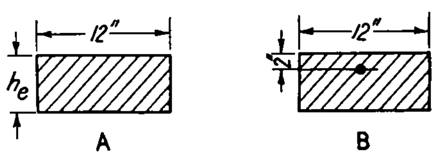


Figure 9

load This is generally considered as being concentrated near the apex of the corner and it is further assumed that, in the vicinity of the corner, the subgrade provides very little if any supporting reaction Under these assumptions the corner must necessarily act as a cantilever subjected to negative bending moment producing tension in the top fibers of the slab

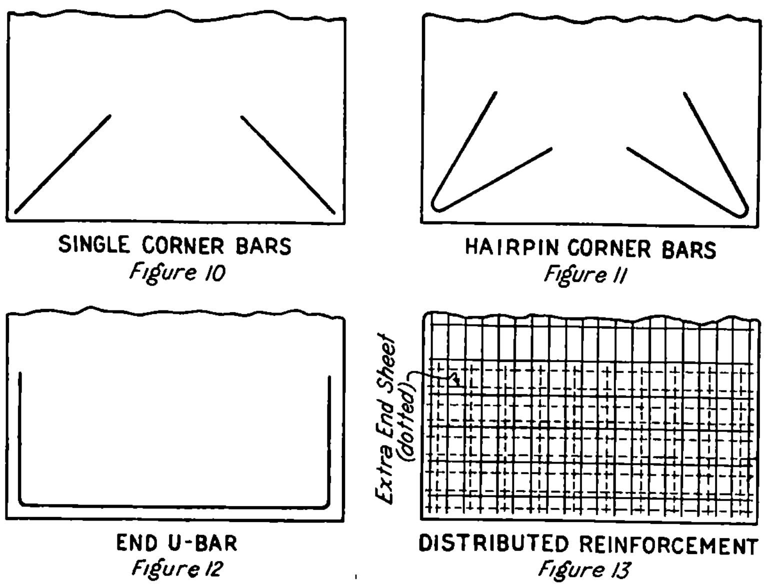
The ideal basis of strengthening a corner would be to reinforce a localized segment of the corner so that it would have adequate strength to carry the maximum applied load as a beam, even though a diagonal corner break should occur Assuming that the corner has been adequately designed as a plain concrete section, this condition of having adequate beam strength after a crack occurs would require provision of corner reinforcement sufficient to convert the section into a reinforced section having the same moment of resistance

Consider, as in Figure 9, two sections A and B, each having the same width and thickness, but section A a plain concrete section, and section B a reinforced section With the reinforcement located 2 inches below the top surface of the slab and taking the internal moment arm as nine-

tenths the effective depth, the moment of resistance of section B is equal to $A_s f_s$ 0 9 $(h_e - 2)$ Equating this to the moment of resistance of section A, which is equal to $2Sh_e^2$, gives,

$$A_s = \frac{2\ 2\ Sh_e^2}{f_s(h_e - 2)} \qquad (6)$$

It is evident that this basis of design will give rather high steel requirements, approximately 0 3 to 0 4 square inches per foot width of section of a nine-inch edge thickness Consistent with this analysis, this heavy sectional area of steel would have to be provided across any corner section and would thus theoretically require an increasing total amount of steel as the distance of the section from the corner increased Obviously it would be a difficult practical matter to provide this, not to mention the amount of steel required It is thus apparent that the strenghtening of slab corners by means of localized reinforcement cannot be determined on any reasonably rational basis of computation with respect to probable bending moments without establishing some definite limit as to probable width of critical section and also without encountering certain practical difficulties in the placing of the required steel



Figures 10 to 13. Methods of Strengthening Slab Corners

Practical design procedure, with respect to corner strengthening, would therefore seem to be to provide a reasonably adequate edge thickness of the plain concrete section to start with and to prevent it from acting as a free independent corner by providing slip-dowels at open

transverse joints. In case additional precautionary strengthening is desired, this may be accomplished by arbitrarily using a moderate amount of corner steel, positioned either as diagonal or marginal members, under the assumption that it will serve as a reasonable protection to the corner against isolation of the corner segment in case a corner break actually occurs.

Figures 10 to 13 illustrate various arrangements of corner reinforcement commonly used. Arrangements indicated by Figures 10 to 12 have the common objection of poor distribution of steel with respect to any diagonal corner section. To eliminate this objection extra end sheets of fabricated reinforcement are often used, thus providing a greater number of small members closely spaced (figure 13). This arrangement also provides much better protection to the small corner segment itself, thereby increasing its resistance against secondary breakage.

Use of Slip-Dowels. The structural function of a dowel is to render a non-interlocking joint shear resistant. The doweling member is thus intended to provide a means whereby the two slab edges forming the joint will deflect simultaneously, rather than independently, under the action of a load applied on one edge only. In performing this function the dowel enables both slabs instead of one to carry the load and is accordingly referred to as accomplishing "transfer the load" across the joint. In order to cause equal deflection of adjoining slab edges, the necessary load to be transferred by a dowel, or group of dowels, is obviously equal to half the applied load.

The dowel, at best, is a somewhat questionable means of rendering joints shear resistant, owing to bending in the dowel itself and the largely uncertain intensity and distribution of its bearing on the concrete. Therefore, in the case of joints which are to remain permanently closed, there is no object in using dowels, since such joints can usually be rendered shear resistant in a more definite and effective manner by utilizing the principle of face interlock such as is provided by tied joints of the tongue-and-groove or dummy type. However, in the case of joints where direct face interlock can not be maintained, some mechanical means of bridging the gap between the faces of the joint must necessarily be provided if a shear resistant joint is desired. Some type and arrangement of steel doweling members therefore appears to constitute the only practical expedient at the disposal of the engineer for accomplishing this objective.

In order that slip-dowels may accomplish any material strengthening of joint edges it is essential that the total number of dowel bars be sufficient to effect a transfer across the joint of half the total maximum joint loading without exceeding the load-transfer capacity of any one dowel. Furthermore, maximum dowel spacing must, in any event, be restricted so as to accomplish a reduction of at least 50 per cent in the bending moment induced in the loaded edge.

RATIONAL DESIGN OF SLIP-DOWELS

Structural Requirements As utilized in joints of the free type, dowels, although serving as stiff rigid members to resist shear, must not offer any resistance to the lateral movement of adjoining slabs This restriction makes it necessary to render the dowels bondless in at least one of the slabs which they connect and, in the case of expansion joints, to provide also a suitable clearance pocket at the bondless end of each bar in order to prevent any thrust against the end of the dowel when the joint closes Obviously the slip feature of the dowel is essential to its proper functioning The end-clearance feature, as required at expansion joints, is, however, not required at contraction joints

End clearance pockets are obtained by encasing one end of the dowel in a tight-fitting tube, the closed end of which is held away from the end of the bar during construction to provide the necessary end pocket in the concrete

In doweling a pavement joint by means of a series of individual steel bars, design procedure, comprises determination of the required diameter, length and spacing of bars as influenced by the magnitude of maxi-

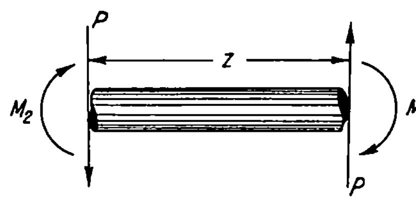


Figure 14

mum wheel load for which the pavement is designed Three conditions of stress must be investigated, (a) transverse shear on the bar, (b) bending in the bar, and (c) bearing of the bar on the concrete

External Forces Consider first that part of the bar extending across the joint opening As in Figure 14, this unit is acted upon by the shearing forces P and the two moments M_1 and M_2 Equilibrium of the unit requires the condition that $M_1 + M_2 = Pz$ If the bar has the same condition of embedment in each slab, then this fact, together with the other conditions of symmetry, would justify the assumption that $M_1 = M_2$. The appropriateness of this assumption, however, is dependent upon the method used in rendering one end of the bar bondless If this is accomplished by the use of a long encasing tube extending to the face of the joint, it is very probable that the bar would not be securely fixed at that face, a condition tending to create a smaller moment than at the other face where a greater degree of fixedness exists By using merely a short capping tube to obtain end clearance and breaking the bond by painting and oiling, a method now generally considered as the best practice, embedment conditions in the vicinity of the joint faces would

be substantially the same in both slabs and the most reasonable assumption as to relative moment values would be $M_1 = M_2$ Assuming this condition to exist and calling M_1 the bending moment at either face of the joint, we obtain

$$M_1 = \frac{Pz}{2}$$

It is next necessary to investigate the magnitude and distribution of pressure on the embedded parts of the bar These portions of the bar may be considered as being tightly surrounded by a material of yielding character and therefore offering resistance to either positive or negative deflection of the bar According to an analysis developed by Timoshenko (7) for somewhat similar conditions, the general expression for deflection of the bar, if the bar has infinite length, gives, when plotted, a wave-curve having gradually diminishing amplitude as the distance from the applied load increases Assuming that the supporting medium is an elastic material, it would follow that the intensity of pressure on the bar at any point is proportional to the deflection at that point, and the intensity of pressure would therefore vary in accordance with a wave-curve having similar characteristics Both the length and amplitude wave are functions of the relative stiffness of bar and foundation This factor of course involves the section modulus of the bar, its modulus of elasticity, and a quantity representing a property of the foundation material which may be termed the "modulus of foundation reaction"

In attempting to apply the Timoshenko analysis to the case of a dowel bar, it is apparent that certain practical considerations introduce elements of some uncertainty First, we must deal with a bar of finite rather than infinite length However, in view of the marked decrease in pressure intensity as the distance from the joint face increases, the effect of this departure from the theory would no doubt be relatively unimportant Secondly, the foundation modulus for concrete, under the condition of a load applied by a small bar embedded in a thin slab which is itself supported by a yielding material having still different elastic properties, is not readily susceptible of evaluation Notwithstanding these uncertainties, the Timoshenko analysis appears useful as a means of indicating a very probable relationship between pressure intensities at various points along a dowel bar and, with certain slight modifications, leads to the assumption of what appears to be a very probable pressure diagram which, with certain approximations, permits of analysis by simple principles of statics

According to Timoshenko, the pressure diagram on half the bar has, in the vicinity of the applied load, approximately the properties indicated in Figure 15 Considering the length of bar covered by the first ful cycle of positive and negative pressure, it is observed that positive pressure occurs over approximately the first four-tenths of each embed-

ded length, negative pressure over the remaining six-tenths, and that the maximum intensity of negative pressure is only about 4 2 per cent of the maximum positive pressure It is also observed that the intensities of both positive and negative pressures correspond very closely to straight-line distributions as indicated in Figure 15 by the superimposed dotted-line diagram

Evidently pressures existing beyond the distance X are extremely small and, for all practical purposes, their effect may be neglected If we thus impose the condition that half the bar length shall be approxi-

Relative Pressure Values

Distance from Load	Pressure Intensity
0	1 000
1X	793
2X	448
3X	177
4X	024
5X	− 035
6X	− 042
7X	− 030
8X	− 016
9X	− 005
1 0X	0 000

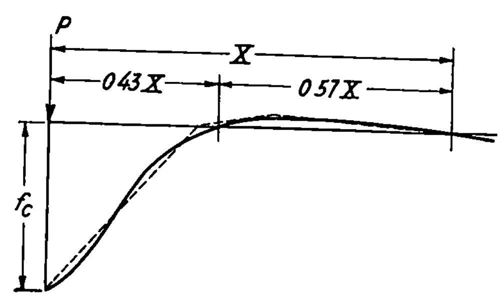


Figure 15 Distribution of Pressure on Embedded Bar of Infinite Length Caused by Load Applied at End. (According to Timoshenko)

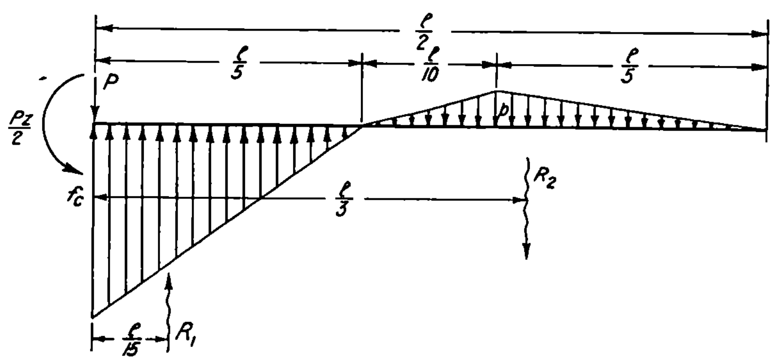


Figure 16. External Forces Acting on Embedded End of Dowel Bar

mately equal to the length covered by the first positive and negative pressure cycles, then, the general characteristics of the pressure diagram for one embedded end of the dowel might be assumed to be in general accord with the curve of Figure 15 The question then arises as to what practical effect, if any, this finite length of bar will have upon the characteristics of the pressure curve as represented by Figure 15

It is probable that, by breaking the infinite continuity of the bar, a slight negative pressure might even exist at the end of the bar due to a tendency to "kick-up" so to speak With dowel lengths of the order

generally used and in view of the relatively small intensity of negative pressure, any end pressure would necessarily be very small and of slight practical significance It appears reasonable, therefore, to assume that the point where the pressure changes from positive to negative is located at a distance from the joint face equal to two-tenths of the embedded length Thus, as in Figure 16, one side of the bar may be removed and the remaining embedded portion considered as a bar acted upon at one end by the shear P and the couple $\frac{Pz}{2}$ and reactions of variable intensity distributed as shown

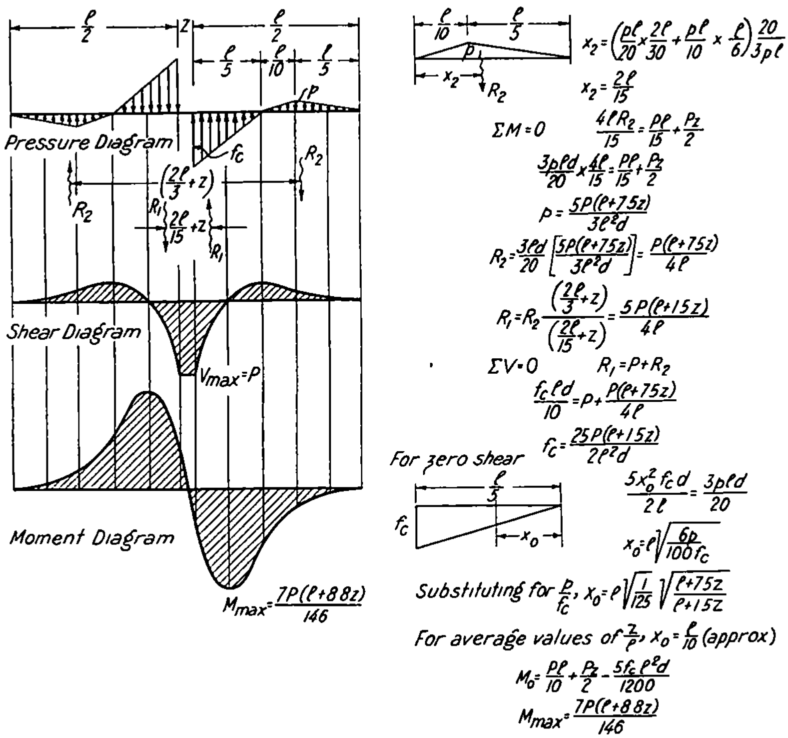


Figure 17. Moments and Shears on Dowel Bars

Considering the pressure distributed as in Figure 16, and calling R_1 the resultant of positive or upward pressure, and R_2 the resultant of negative or downward pressure, we have,

$$R_1 = \frac{f_c l d}{10} \quad \text{and} \quad R_2 = \frac{3\,pld}{20}$$

The condition $\Sigma M = 0$ gives,

$$p = \frac{5\,P(l + 7.5\,z)}{3\,l^2 d}$$

and the condition $\Sigma V = 0$ gives,

$$f_c = \frac{25\ P(l + 1\ 5\ z)}{2\ l^2d}$$

Moments and Shears. Obviously the maximum shear occurs at the face of the joint and is equal to P. Zero shear occurs at a distance x from the joint face when,

$$x = \frac{l}{5} - l\sqrt{\frac{1}{125}}\sqrt{\frac{(l + 7\ 5\ z)}{(l + 1\ 5\ z)}}$$

or, for average ratios of z/l, very nearly, $x = \frac{l}{10}$.

Using this approximate position of zero shear, maximum bending moment in the bar is,

$$M_{\max} = \frac{7\ P(l + 8\ 8\ z)}{146}$$

Load-Transfer Capacity. In developing the following expressions for load-transfer capacity, as limited by various conditions of stress, the dowel is, in each case, assumed to be a *round bar,* since this is the shape universally used for dowel bars.

Equating the shearing resistance of the bar to P, the load-transfer capacity, as limited by *shear on the bar,* is,

$$P = 0.785d^2f'_s \tag{7}$$

Equating the moment of resistance of the bar to the maximum bending moment, the load-transfer capacity, as limited by *bending in the bar,* is,

$$P = \frac{2\ d^3f_s}{(l + 8\ 8\ z)} \tag{8}$$

Transposing the above expression for f_c, the load-transfer capacity, as limited by *bearing on the concrete,* is,

$$P = \frac{f_cl^2d}{12\ 5\ (l + 1\ 5\ z)} \tag{9}$$

Length of Dowels Comparison of formulas (8) and (9) indicates that, on this basis of analysis, increase in dowel length increases its load-transfer capacity in bearing but decreases its capacity in bending By equating equations (8) and (9), the limiting dowel length, for equal capacity in both bearing and bending, is

$$l = d\sqrt{\frac{25\ f_s}{f_c}}\sqrt{\frac{(l + 1\ 5\ z)}{(l + 8\ 8\ z)}} \tag{10}$$

or, for average ratios of z/l, approximately

$$l = d \sqrt{\frac{20 f_s}{f_c}} \tag{10a}$$

The length l, thus determined, is the total embedded length. The actual length of dowel bar is equal to $l + z$

Spacing of Dowels. If the ends of two slabs are adequately doweled together, then each slab end, taken as a whole, will in effect, have to carry only half the total of any system of loads that might be simultaneously applied at the joint, even though they all be applied at the edge of one slab only If there were no intervening free spaces between the doweling members, then the maximum bending moment produced in the edge of the loaded slab would be only half as much as it would be if it did not receive any assistance from the other slab However, when doweling is provided by individual bars, spaced at certain intervals across the slab width, there are intervening spaces between dowels in which the loaded slab edge must necessarily act as an independent edge for loads applied between the dowels

The bending moment produced by a concentrated load applied at the edge of a pavement slab is a maximum directly under the load and decreases very rapidly as the distance from the load increases. Westergaard (6, 8) has shown theoretically that the moment is approximately half the maximum at a distance of only 9 to 15 inches from the load, depending upon the thickness of slab and modulus of subgrade reaction Therefore, if dowels are placed more than 18 to 30 inches apart they are ineffective in providing the desired 50 per cent moment reduction in the slab edge when a load is placed midway between dowels, regardless of the load-transfer capacity of the dowels themselves For the case of a seven-inch slab and moderately soft subgrade conditions ($k = 50$), Westergaard (8) shows that dowel spacing should not exceed approximately 24 inches For stiffer subgrade conditions maximum permissible spacing would be even less (about 18 inches with $k = 200$) It is therefore evident that a reasonably close spacing of dowels, probably not more than 24 inches in any case, is absolutely necessary if dowels are to be expected to contribute any appreciable reduction in the edge stress occurring at free joints

This maximum limitation in spacing is entirely separate and apart from any spacing requirements as may be determined by the load-transfer capacity of the dowels themselves It simply means that, regardless of the size of dowels used, the slab edge, insofar as the effect of concentrated edge loading is concerned, will function as a free edge if dowels are not spaced close enough together to fall well within the zone of positive stress as caused by a concentrated load applied on one edge midway between dowels This phase of dowel spacing, of course, has no significance in the case of slabs of uniform thickness or even in thin-center slabs

where other means of edge strengthening, such as a bottom edge bar, is employed, It is, however, of prime importance in the case of thin-center slabs where full dependence for edge strengthening is placed upon the dowels

Another and wholly independent phase of dowel spacing is the maximum permissible spacing as determined by the load-transfer capacity of a single dowel Obviously, all the dowels in the full pavement width must be capable of transferring half the total of all maximum wheel loads that can be simultaneously applied at the joint The total number of dowels must therefore be at least equal to half the sum of all loads applied to the joint divided by the load-transfer capacity of a single dowel, and actual spacing at any point thus depends upon the distribution of the applied loads as among the several dowels

In the case of a two-lane pavement the maximum condition for load concentration at the joint would occur with two passing vehicles of the heaviest allowable wheel load type Thus it would be possible to simultaneously concentrate four maximum wheel loads at the joint These

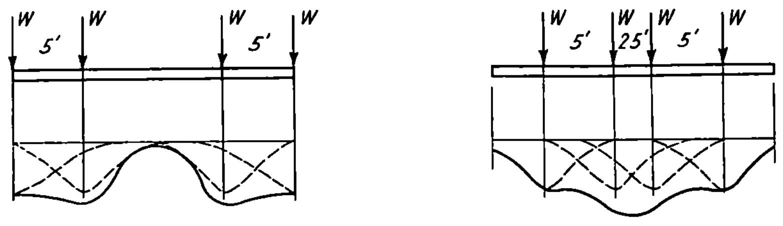


Figure 18

loads would necessarily be applied in pairs, the loads of each pair having a fixed spacing of approximately 5 feet Two conditions of relative placement are suggested as in Figure 18, (a) each vehicle traveling as close to the edge of the pavement as possible, and (b) each traveling close to the center, say with a clearance of 2 5 feet between inside wheels Considering the slab end as a whole the general character of deflection produced by each load would be as indicated by dotted lines in figure 18 (6, 8) By the principle of superposition it is seen that case (a) would give a fairly uniform distribution of two-wheel loads over a pavement width of approximately eight feet, and case (b) a fairly uniform distribution of four wheel loads over a width of approximately 16 feet For the case of an 18-foot or even a 20-foot pavement, this would indicate that under maximum joint loading the end of the slab would have a reasonably uniform deflection across its entire width and it would therefore be reasonable to assume two wheel loads as uniformly distributed across a lane width Accordingly, the applied load per inch length of a joint would be $\frac{W}{60}$ pounds for a ten-foot lane and $\frac{W}{50}$ pounds for a nine-foot

lane, half of which, in either case, must be transferred across the joint by the dowels

Calling s the spacing of dowels in inches and P the load-transfer capacity of a single dowel, we have

for 20-ft pavements,

$$s = \frac{120\,P}{W} \tag{11}$$

for 18-ft pavements,

$$s = \frac{108\,P}{W} \tag{11a}$$

Allowable Unit Stresses In applying rational stress analysis to dowels there appears to be no established precedent as to appropriate allowable working stresses While the same fundamental considerations, that pertain to the selection of a proper working stress for any material of known strength, no doubt apply, still certain conditions, peculiar to the dowel, suggest the possibility of some departure from customary conceptions of working stress limitations In the case of bending in a dowel bar, for instance, should the same unit stress restriction be applied as in the case of direct tension? According the common practice, this would establish a working stress of approximately 20,000 pounds per square inch for bars of intermediate grade steel. If this stress is safe for direct tension, under which every fiber of the cross section is stressed to the maximum, certainly it would seem unduly conservative for the case of bending whereby only the extreme fibers are stressed to the maximum, and especially so in the case of a circular cross section wherein only one fiber on either side of the bar is stressed to the theoretical maximum Even from the standpoint of stress in the compression side there can be no great concern, since a dowel, although a relatively slender member, is securely restrained against buckling in any direction

These conditions, which would appear to be favorable to the use of a somewhat higher fiber stress in bending than in direct tension, are however somewhat offset by the fact that immediate reversal of stress accompanies every application of the maximum bending stress in the case of a dowel bar Considering the fatigue limit of common steels under reversal of stress as being about 75 per cent of the yield point, it would appear that in the case of dowels the allowable unit extreme-fiber stress could be taken at approximately 75 per cent of the yield point for the particular grade of steel used This would correspond to an allowable unit stress for bending in dowels of approximately 30,000 pounds per square inch for intermediate grade, and approximately 40,000 pounds per square inch for hard grade steel

Unit bearing of the bar on the concrete should be conservatively limited in order to prevent undue crushing of the concrete at the face of

the joint where the stress is a maximum Some slight surface spalling will undoubtedly occur in any case, but the intensity of bearing stress decreases very rapidly as the distance from the face of the joint increases and, even with a fairly high computed intensity at the extreme edge, the effective maximum intensity over an appreciable area would be materially less than the maximum computed value In other words, at the extreme edge, the bar would tend to seat itself over a small width and, once this condition were established, it would seem that no progressive crushing of the concrete would develop if the computed maximum unit stress is held within reasonable limits

Allowable unit compressive stress on the extreme fiber, in the case of concrete beams in building work, is usually prescribed as 40 per cent of the 28-day compressive strength of the concrete The quality of concrete commonly used in pavements will, in general, develop 28-day compressive strengths of from 3,000 to 4,000 pounds per square inch On the basis of 40 per cent, this would correspond to an allowable unit bearing stress of from 1200 to 1600 pounds per square inch It would

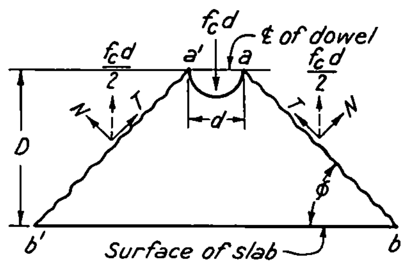


Figure 19

thus appear that, in designing dowels, a unit bearing stress of approximately 1200 pounds per square inch could be utilized in almost any case In the case of concrete that may be considerably better than the average, the allowable bearing stress could probably be increased to 1500 pounds per square inch or even more

Required Concrete Covering for Dowel Bars Dowel bars must be located a sufficient distance from both the top and bottom surfaces of the slab to afford adequate protection against breaking through the concrete under the "prying" or "lever-like" action of the dowel For the case of dowels placed at the center of the slab, as is common practice, this requirement establishes a limiting maximum diameter of dowel bar permissible in a slab of given thickness

The amount of force per unit length of bar tending to tear out a segment of concrete either above or below the dowel is equal to the unit bearing pressure times the diameter of the bar This is a maximum at the face of the joint and decreases as the distance from the joint face increases. Consider, as in Figure 19, the segment *abb'a'* taken at the

face of the joint and having a thickness of one inch Neglecting shear on the vertical face of the segment, the vertical force $f_c d$ applied by the dowel, is resisted by the resultant stresses on the sections ab and $a'b'$ Assuming conditions of symmetry with respect to the two inclined sections of resistance, the resultant stress on each section is equal to $\frac{f_c d}{2}$ This force may be resolved into two components, N normal to and causing tension on the section and T tangential to and causing shear Owing to the relatively low tensile strength of concrete, tension on the section is the limiting case Thus we have

$$N = \frac{f_c d \cos \phi}{2}$$

and the area of section ab (1 inch wide) is

$$A_{ab} = \frac{D}{\sin \phi}$$

Calling f_t the unit tension on section ab, we have

$$f_t = \frac{N}{A_{ab}} = \frac{f_c d \sin \phi \cos \phi}{2\,D}$$

Maximum unit tensile stress thus occurs when $\sin \phi \cos \phi$ is a maximum, or when $\phi = 45°$, and for this value of ϕ, the product $\sin \phi \cos \phi$ has a numerical value of 0 50 Therefore we have, for maximum tensile stress in the concrete,

$$f_t = \frac{f_c d}{4\,D}$$

or

$$D = d\left(\frac{f_c}{4\,f_t}\right) \tag{12}$$

In practice, f_t should be limited to about one-twelfth f_c Substituting $f_c = 12 f_t$ in the above expression for D we have

$$D = 3d \tag{12a}$$

This would indicate that the center of a dowel bar should in general be located at least three bar diameters from the nearest slab surface, or, when located at the center of the slab section, the diameter of a dowel bar should not exceed one-sixth the thickness of the slab

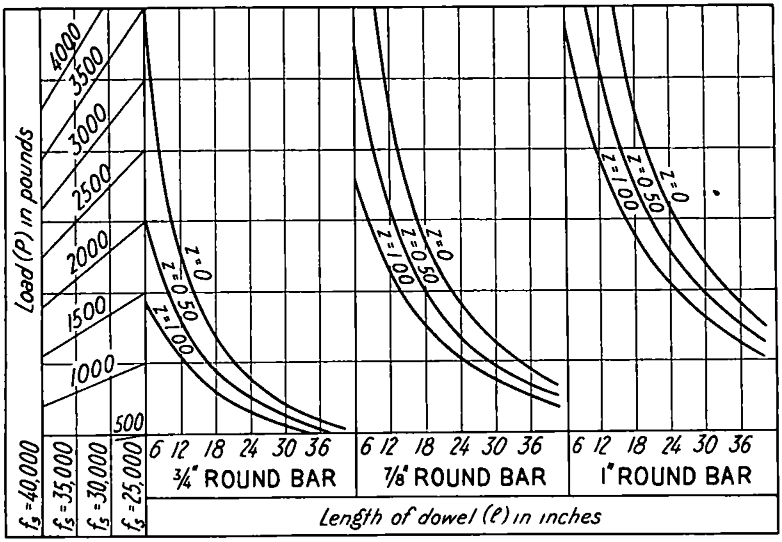


Figure 20. Load-Transfer Capacity of Round Dowel Bars as Limited by Bending. Based on Formula (8)

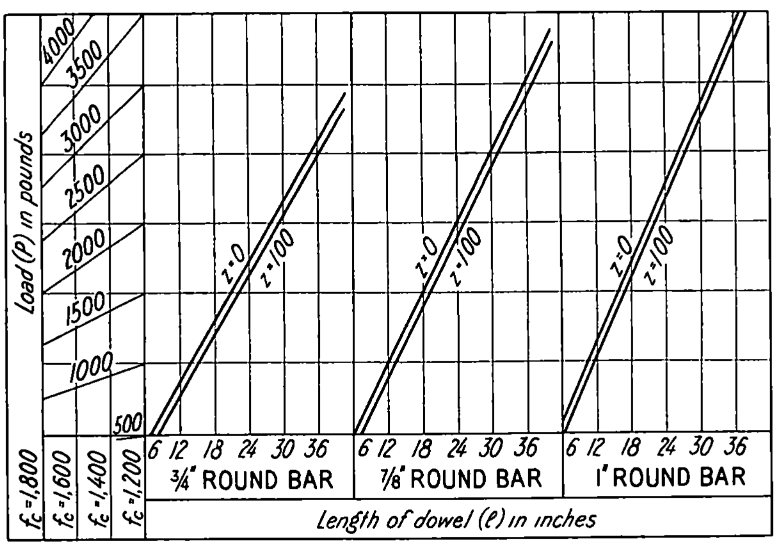


Figure 21 Load-Transfer Capacity of Round Dowel Bars as Limited by Bearing. Based on Formula (9)

Numerical Example of Dowel Design

Given Conditions

Pavement Width, 20 ft

Slab Length, 40 ft ($\frac{3}{4}''$ expansion joints @ 80 ft, dummy contraction joints @ 80 ft, alternating with expansion joints)

Maximum Wheel Load, 8000 lbs

Impact Allowance; 25%

Allowable Stresses,

$f_s = 30{,}000$ lbs per sq in (bending)

$f_c = 1{,}200$ lbs per sq in

Computations

Assume 80-degree temperature range below temperature at time of construction

$$z = 75 + (0000055 \times 80 \times 40 \times 12) = 96''$$

$$l = d\sqrt{\frac{20f_s}{f_c}} = d\sqrt{\frac{20 \times 30000}{1200}} = 22\ 4d$$

Using $\frac{3}{4}''$ round bars

$$l = 22\ 4 \times 75 = 16\ 8''$$

Actual dowel length $= l + z = 17\ 8''$

$$P = \frac{2d^3 f_s}{(l + 8\ 8z)} = \frac{2 \times 75^3 \times 30000}{16\ 8 + (8\ 8 \times 96)} = 1000\#$$

$$s = \frac{120P}{W} = \frac{120 \times 1000}{1\ 25 \times 8000} = 12''$$

Use $\frac{3}{4}''$ round, 18″ long, spaced 12″ (say 20 dowels)

Using $\frac{7}{8}''$ round bars

Same procedure requires,

$\frac{7}{8}''$ round, 21″ long, spaced 16″ (say 14 dowels)

Using 1″ round bars

Same procedure requires,

1″ round, 24″ long, spaced 23″ (say 10 dowels)

Comparative Costs

Unit costs

Dowel cost, 3¢ per lb (delivered, oiled ready to place)

Cost to place, 10¢ per dowel (including end cap)

Cost per Joint

$\frac{3}{4}''$ round,	dowels, 20 × 2 25#	@ 3¢ =	\$1 55	
	placing, 20	@ 10¢ =	2 00	\$3 35
$\frac{7}{8}''$ round,	dowels, 14 × 3 60#	@ 3¢ =	\$1 50	
	placing, 14	@ 10¢ =	1 40	\$2 90
1″ round,	dowels, 10 × 5 35#	@ 3¢ =	\$1 60	
	placing, 10	@ 10¢ =	1 00	\$2 60

Typical Present Dowel Design

$\frac{3}{4}''$ round, 30″ long, spaced 32″

Cost per joint

dowels, 8 × 3 75	@ 3¢ =	\$ 90	
placing, 8	@ 10¢ =	80	\$1 70

Increased Cost

40-ft slab length gives 132 joints per mile

Improved Design	132 @ \$2 60 =	\$343 per mile
Typical Present Design	132 @ 1 70 =	224 per mile
	Increased Cost	\$119 per mile

SUMMARY CONCLUSION

Edge and Corner Strengthening Edges of free type joints may be strengthened by, (a) thickening the edge sufficiently to be adequate, without marginal steel, as a free independent pavement edge, (b) by reinforcing the joint edge with bottom tensional steel, and (c) by the use of slip dowels

(a) Edge thickening, although applicable in structural principle to either longitudinal or transverse joints, has proved feasible and satisfactory when applied to longitudinal joints but has not proved satisfactory for transverse joints because of a marked tendency to promote transverse cracking at or near the section where thickening begins

(b) Edge strengthening of transverse joints by means of tensional steel is practical and satisfactory The procedure in applying this method consists essentially in considering the joint edge to be a beam approximately 12 inches wide requiring reinforcement sufficient to render its moment of resistance equal to that of an equivalent width of the unreinforced longitudinal edge section Consistency of analysis on this basis would require that such steel be located in the bottom of the slab, except in the vicinity of the exterior corners as formed by the joint where corner strengthening would require its location in the top of the slab

(c) When full reliance is placed upon slip-dowels as a means of strengthening joint edges, they should be consistently proportioned as to size and spacing on the basis of their load-transfer capacity as limited by safe bending in the dowel or by its bearing on the concrete Maximum dowel spacing must also be limited in such a way as to effect a proper reduction of the bending moment in the loaded edge of the joint, this limitation as to maximum spacing being independent of that required for adequate transfer of load across the joint

Slip Dowels The general purpose of slip dowels as utilized in concrete pavements is to enable the two edges formed by an open joint to deflect simultaneously rather than independently as traffic loads pass over the joint In performing this function two edges instead of one will carry the loads applied at a joint and each joint edge will accordingly be stronger than it would be if it were compelled to carry the applied loads without assistance from the other edge It is fully realized that this doweling of slab edges across an appreciable gap, by means of steel members of comparatively small diameter and having relatively wide spacing, is by no means an ideal structural condition

If strengthening of the joint edge were the only objective, dowels could be dispensed with and more desirable methods of edge strengthening employed, but the strengthening of joint edges at an open joint unfortunately is only a part of the problem If it is desired, as it should be, to maintain an equal surface elevation on both sides of the joint at all times, then some mechanical connection between the joint faces

must be provided Thus it may be concluded that some mechanical means of bridging the gap between the faces of an open joint is desirable, not only with respect to the actual structural strength of the joint edges, but also as a means of maintaining at all times an even surface elevation at the joint Dowels appear to be the simplest means at the disposal of the highway engineer for accomplishing this objective

Current practice in the use of dowels indicates very clearly that the selection of dowel details has been and now is pretty much a hit-and-miss procedure Certain tests, although limited in scope and character, have indicated that many of the dowel arrangements now commonly used are practically useless from the standpoint of their truely intended purpose This is indicated rather vividly by the Slack Tests (9) which showed that $\frac{3}{4}$-inch round bars spaced 32 inches apart and crossing a $\frac{1}{2}$-inch expansion joint were only about 13 per cent efficient in doing what they really were supposed to do With very few exceptions this particular dowel arrangement can be taken as typical of dowel details now in common use

These test results are closely substantiated by application of the rational analysis herewith suggested This analysis leads to the general conclusion that the $\frac{3}{4}$-inch dowel is practically useless as a load-transfer medium unless spaced at very close intervals, probably 8 to 10 inches This suggests the possible necessity of discontinuing the use of the now popular $\frac{3}{4}$-inch dowel and resorting to the use of $\frac{7}{8}$-inch or even 1-inch bars Furthermore, the analysis, herewith suggested, indicates that an unnecessarily long length of dowel bar serves no useful purpose and may even contribute to high bending stresses, if the assumptions as to pressure distribution are reasonably correct Recourse to shorter length, larger diameter, and or closer spacing thus appears to be the logical basis of approach in attempting to render the dowel more effective in serving the structural purpose for which it is intended

The rational stress analysis of dowels, as herewith suggested, constitutes merely an effort to depart from the present arbitrary basis of deciding as to what the diameter and spacing of dowels should be, and attempt to apply some form of rational analysis of dowel requirements There is no reason why test data, similar to the results shown by the Slack Tests and others, made upon existing dowel arrangements which have been arbitrarily selected, should reflect necessarily upon the dowel as a useful and serviceable instrument for accomplishing a certain desired structural function in connection with the action of pavement joints In passing judgment upon the structural merit of the dowel, it would, therefore, seem that hasty conclusions as to its efficiency should not be based upon random test data or upon the behavior of obviously inadequate designs Such data, instead of serving to discredit the dowel itself, should inspire an effort on the part of highway engineers to improve the standards of dowel design, a precedure which can undoubtedly be approached on some basis of rational computation

Rational vs Arbitrary Design The conditions to which pavement slabs are subjected are, of course, numerous and in some respects extremely complex and uncertain, thus rendering them difficult of evaluation However, with the aid of conservative and appropriate assumptions reasonable and highly probable stress values may be determined by computation

Stress values thus determined, although they may be questionable as to their exact degree of accuracy, will nevertheless serve as consistent indications of stress possibilities and as such may be utilized numerically as an orderly basis for proportioning joint details consistent with the use of materials of known strength. This, of course, involves the use of certain empirical factors and basic assumptions but this necessity is not alone peculiar to the structural features of a concrete pavement, it is, in greater or lesser degree, necessary in the design of any structure that the engineer is called upon to build Many of the important structural details of the concrete pavement slab are amenable to the application of certain recognized basic principles of structural design and a more universal application of those principles will undoubtedly result in a greater uniformity of practice, a more consistent proportioning of slab details, and a general betterment of design standards.

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DISCUSSION

ON

JOINTS IN CONCRETE PAVEMENTS

MR R B GAGE, *New Jersey Highway Department, The Ledge Type of Transverse Joint* Most of the concrete pavements constructed in New Jersey since 1922 have longitudinal and transverse joints The slabs are generally 10 feet wide and 35 feet long

The transverse joint openings are generally from one-half to five-eighths inch in width, the joint filler being of the premoulded type All transverse joints are reinforced, containing from 6 to 12 dowels, 20 to 30 inches long The top of each joint was originally sealed with a hot liquid bitumen and re-sealed in the same manner at frequent intervals after construction A serious effort has been made to keep all joints so sealed that water will not pass through them into or on the subgrade

This type of transverse joint has not been very satisfactory for several reasons

In localities where the vertical drainage of the subgrade is poor and the pavement subject to considerable heavy truck travel, one end of each slab is soon forced below grade The more impervious and denser the subgrade, the quicker and more pronounced the displacement Water soon collects beneath this end of the slab, which accelerates displacement

The front of each slab in the direction of traffic is the end subjected to the greatest impact and load This end is slowly forced out of alignment as the surface of the subgrade beneath it is lowered by erosion, consolidation or compression Since the elasticity of the concrete prevents the slabs from remaining at the maximum depressed position, a narrow opening is soon formed between the bottom of the slab and the surface of the subgrade This opening soon fills with water that is held in place by the shoulders and imperviousness of the subgrade The churning action of the slab caused by traffic, soon makes a mud out of the surface of the subgrade, some of which mixed with water, is forced to the surface through the joints or along the edges of the slab, each time a heavy load passes over it As the opening beneath the slab is thus gradually deepened and lengthened, a corresponding settling of the slab takes place Objectionable bumps which lower the riding properties of the pavement are soon formed at each joint The supporting power of the subgrade being thus reduced, a transverse crack will sooner or later appear from three to ten feet from the joint

Fortunately, only the front end of each slab is thus depressed When a load is applied progressively along a slab, little deflection apparently takes place in the rear end of the slab, but when the entire weight of a

heavily loaded truck travelling at 25 to 30 miles per hour is suddenly transferred across a joint to the adjacent slab, the front end of this slab is forced out of alignment. This end of a slab may be forced from $\frac{1}{4}$ to $\frac{1}{2}$ inch below grade within one year after construction, depending upon the character of the subgrade, density of traffic, and drainage conditions.

Dowels are of little value in preventing such displacement, they are either bent or openings worn in the concrete around them of sufficient size to permit displacement. In some cases both of these conditions have been observed at a given joint.

The difference in the behavior of the ends of adjacent slabs at a given joint is remarkable. The end of one may be floating in a sea of mud, yet the subgrade beneath the adjacent slab end remains perfectly dry and stable. Openings made on the edges of adjacent slabs have shown these conditions. The rear end of a slab is very seldom, if ever, forced below grade. This end is not only able to carry its own weight plus the load applied by traffic, but also the load applied through and by the dowels when the end of the adjacent slab is forced below grade. This added load is frequently sufficient to bend $12\frac{3}{4}$ inch dowels without affecting the stability or elevation of the rear end of the adjacent slab.

The surveys made of concrete pavements where the slabs have gotten out of alignment indicate that the rear ends of the slabs in the direction of traffic do not need any additional support to keep this end of the slab at the required elevation. This end of the slab is seldom, if ever, depressed regardless of the density and type of travel or character of subgrade.

In New Jersey there is a considerable area where the subgrade is a sandy loam, the vertical drainage of which, was thought sufficient to prevent the accumulation of surface waters. With lighter traffic such subgrades would, no doubt, give satisfactory service but such is not the case with the present type of traffic. Consequently, the type of joint to use and drainage system to install to eliminate these adverse conditions, has become a very serious problem.

The dowel type of joint appears to be of little value in keeping adjacent slab ends correctly orientated. Other types of transverse joints have been developed and tried out. Many of these appear efficient but are quite costly. The type of joint shown on Figures 1, 2, 3 appears to be both efficient and inexpensive, the cost being about the same as that of the dowel type of joint. This new type has now been in use for over a year in a locality where the subgrade is a decomposed red shale with very poor vertical drainage and has been subjected to a daily traffic of 2,000 trucks and 20,000 passenger cars. To date they have given very good service and have kept the adjacent slab ends in the required position. Other slabs constructed at the same time in the immediate vicinity where the dowel type of joint was used, have shown a considerable amount of displacement.

The merits of this type of joint depends upon the following factors

1 The rear ends of the slabs are able to carry part of the loads of the front ends of the adjacent slabs

2 Since the total bitumen contained in this joint is only that needed to seal it, the volume of bitumen required is small compared to that used in the ordinary joint, consequently, any longitudinal movement of the slabs will change the elevation of the bituminous seal but very little This reduces to a minimum the usual joint impact due to the accumulation of excess bitumen at the surface of the joint opening

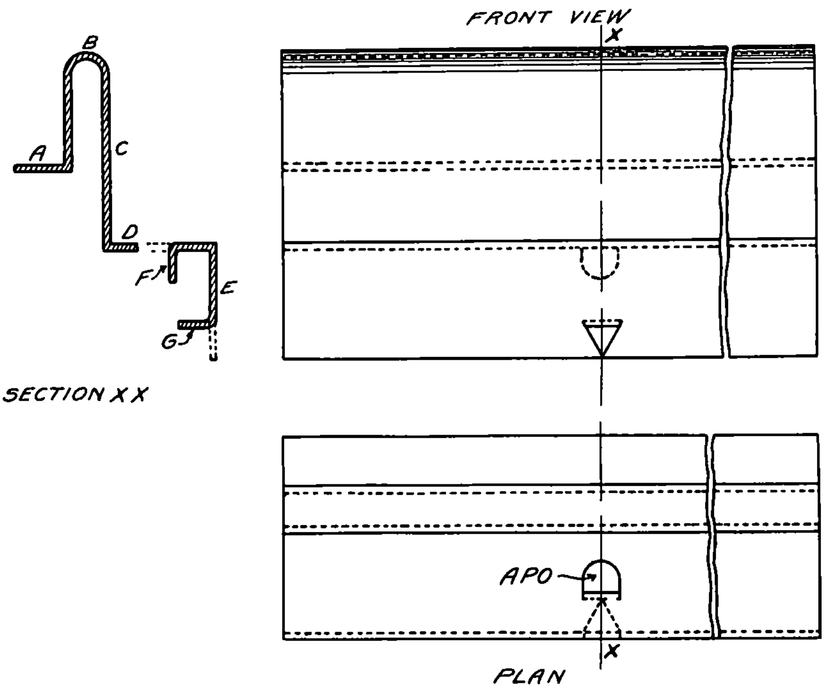

Figure 1. Metal Form for Holding Joint Filler and Forming Partition Walls

3 Unless the rear ends of the slabs are forced below the required elevation, the adjacent ends of the slabs at a given joint cannot get out of alignment

This joint can be given additional strength by reinforcing it, but to date no conditions have developed to indicate that such reinforcement is needed

Figure 1 shows the metal form used to hold the joint filler in place as well as to form the partition walls for the supporting shelf of concrete

Figure 2 shows the complete joint installed on the subgrade ready for the placing of the adjacent concrete

Figure 3 shows the finished joint in place with the top seal coat of bitumen applied

The metal form shown in Figure 1 is made from a single sheet of 22 or 24 gauge galvanized steel and can be made in almost any convenient length. When a pavement is constructed in lanes or with a center joint, this metal form can easily be made of such a length as to equal the width of the slabs The projections F are about ½ inch wide and long and are only intended to hold the joint filler in place. Projections G are triangular barbs that help to hold the joint filler $J. F.$ in position These are stamped out of the joint form itself and are generally spaced from 24 to 36 inches apart, depending upon the rigidity of the joint filler

The joint filler $J. F.$ can be a corrugated paper or a premoulded bituminous mastic, however, the premoulded bituminous mastic should not be used in the top half of the joint unless ample space is left in the

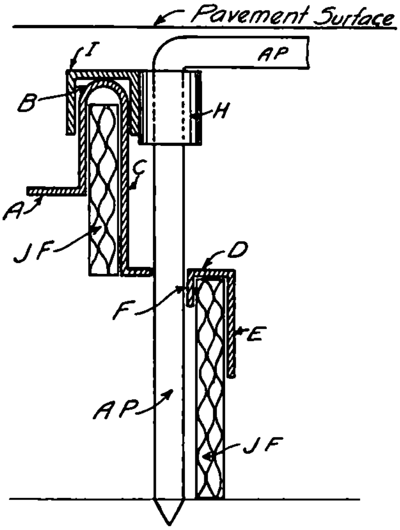

Fig. 2

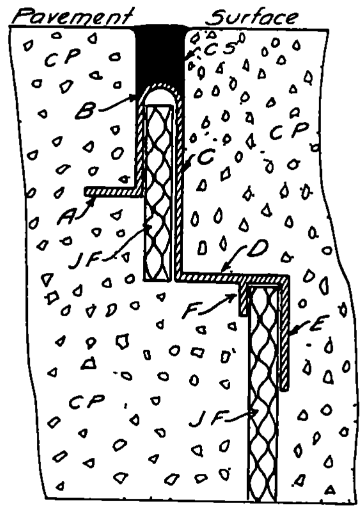

Fig. 3

Figure 2. Joint Installed on Subgrade Ready for Concreting

Figure 3. Finished Joint

top to take care of the reduced volume opening during the expansion of the concrete A premoulded bituminous mastic is not recommended for use in this portion of the joint.

This joint is very easy to prepare and install. The strips of joint filler J F are inserted in the metal form as shown in Figure 2. The joint is then placed on the subgrade and the anchoring pins A P driven through the openings A P O (Figure 1).

This anchoring pin contains a sleeve H which holds the top of the metal form rigidly in place, while the bottom portion of the joint filler J F is held in the required position by both the anchoring pin $A. P$ and the vertical projection of the metal form E. The width of the vertical projection of the metal form, designated as E, can be increased

or decreased according to the thickness of the metal and the stiffness of joint filler Usually this projection should extend down about one-quarter of the thickness of the pavement The projecting shelf designated as *D* is usually placed in the center of the pavement and the top of the metal form at *B* should be not less than one inch below the surface of the pavement.

This joint is sealed by both the metal form and the bituminous capping, since the metal form is anchored in the concrete at *A* and sealed on the surface *D* and the bituminous seal *C S.* (Figure 3) further seals the joint by adhering to the adjacent concrete

It is to be noted that there is very little bitumen used as the cap seal in this joint, consequently, the usual movement in the pavement slabs during expansion and contraction will change the volume of this bitumen but very little, therefore, the bitumen will not be pushed up and form a bump in hot weather nor a depression in cold weather, requiring the joints to be re-poured late in the Fall.

In passing judgment on the merits of this type of joint, the fact should not be overlooked that the rear ends of the slabs in the direction of traffic of pavements ten years old or more have not been forced out of alignment regardless of subgrade conditions, type or character of traffic.