PAPER No. 181.

PROVISION FOR TEMPERATURE AND SHRINKAGE MOVEMENTS IN CEMENT CONCRETE STRUCTURES (PLAIN AND REINFORCED).

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1. The Bugbear in Concrete Construction.

Reinforced concrete appeals to every Engineer and Architect as the most suitable material of construction in every field. In India, however, its use, though steadily increasing, has had a rather limited sphere on account of India being a land of climatic extremes. Many a costly structure has, in the past, been disfigured by unsightly cracks. In a few cases, even the very stability of the structure has been endangered. Yet most of these cracks can be avoided by judicious methods of construction and making proper provision for expansion and contraction, based on an intelligent understanding of the factors, which are responsible for such cracks.

2. Scope of the Paper.

There are four factors producing alterations in length of concrete structures, namely, variation in temperature, variation in moisture content or humidity, creep under load and change in the modulus of elasticity of the material. The last two factors have a great bearing on the distribution of internal stresses and consequently on the design of reinforced concrete structures, but so far as the total movements of the structures are concerned, their effects are, practically, completely masked by the movements caused by temperature and humidity variations. It is, therefore, proposed to confine the discussion in this paper only to the effects of the first two factors.

An attempt has been made to show that provision for expansion and contraction in cement concrete structures, whether plain or reinforced, is the only practical remedy against unsightly cracks and water leaks. A brief description of joint fillers, types of joints, and arrangements for expansion at bridge ends has been included in order to review the current practice, within the compass of the Author's knowledge.

Beyond discussing the fundamental principles, it has not been possible to suggest suitable locations for joints ; this would have involved a study of cracks in the existing structures. Nor has the design of suitable reinforcement, required for temperature and shrinkage, been discussed in sufficient detail. Arched members, too, have been deliberately left out; these can be designed to take care of contraction or expansion to a certain extent by elastic deformation or by introduction of temporary hinges. Any of these subjects is important enough to warrant a separate paper for itself.

3. Effects of Temperature Variation.

(*i*) *Linear Change in Dimensions.*—The coefficient of expansion for stone concrete 1 : 2 : 4 is 0·000,006,6 (according to tests[1] made, in the College of Civil Engineering, Cornell University, in 1916). The variation of temperature in India, from winter minimum to summer maximum, may be from 32° Fah. to 190° Fah. Assuming a more conservative figure of 135 degrees Fah. for the annual temperature variation, change in length of a concrete structure would be 1·07 inches per hundred feet, or 53 inches per mile.

(*ii*) *Stress Produced in Plain Concrete.*—If the concrete structure is prevented from expanding or contracting in any way, a stress equal to the change effected in length by temperature multiplied by the modulus of elasticity of concrete would be set up in the concrete.

The modulus of elasticity for an ordinary 1 : 2 : 4 mix is 2,000,000 pounds per square inch.

$$\left.\begin{array}{c}\therefore\text{tensile stress per degree (Fah.) fall}\\ \text{or}\\ \text{Compressive stress per degree (Fah.)}\\ \text{rise in temperature.}\end{array}\right\}\begin{array}{l}=0{\cdot}000{,}006{,}6\times 2{,}000{,}000\\ =\text{say, 13 pounds.}\end{array}$$

Assuming the tensile strength of concrete (1 : 2 : 4) equal to 140 pounds per square inch (see Appendix A for tensile strengths and moduli of elasticity for various mixes), a fall of about 11° Fah. in temperature would be sufficient to crack this concrete, unless provision exists for the concrete structure to move, before the tensile stress reaches the breaking point. The forces which tend to prevent this movement may be either structural or frictional.

Structural obstructions are avoided by providing suitable contraction and expansion joints. Frictional forces are kept within safe limits by suitable spacing of such joints or by providing special bearings under one or both ends of the concrete structure to allow easy movement. These are discussed in detail later on.

(*iii*) *Stress Produced in Reinforcing Steel.*—If the temperature reinforcement bars are continuous for such a length that the ends may be considered as practically immovable, a drop in temperature has a tendency to shorten the bars, and produces a tensile stress which is independent of the distance between the restrained ends.

The coefficient of expansion[1] of steel, is 0·000,006,5 per degree Fahrenheit and the modulus[1] of elasticity of all grades and kinds of steel is about the same and is usually taken as 30,000,000 pounds per square inch in both tension and compression. Therefore, stress per degree fall of temperature = $30{,}000{,}000 \times 0{\cdot}000{,}006{,}5 = 195$ pounds per square inch. For a drop in temperature of 50 degrees from the normal, a tensile stress of 9,750 pounds per square inch is produced in the longitudinal reinforcing bars, provided there is no relative movement between the superstructure and its bearing support. Tests[3], however, (made in U. S. A. experiencing extremes in climate similar to India) indicate that this temperature stress rarely exceeds 6,000 pounds per square inch.

As the maximum live loads seldom come over the structure during the coldest weather, it is considered unlikely that the maximum stress due to temperature, will occur at the same time, with the maximum stress due to loads. Thus, generally, where no proper provision for contraction is made, a stress in steel of 12,000 pounds per square inch is used, allowing 4,000 pounds per square inch to take care of stresses produced by fall in temperature[3], before the allowable working stress in steel is reached.

By the use of an expansion device at one end of the superstructure, the temperature stresses may be eliminated and the full working stress of 16,000 pounds per square inch may be used in the design.

4. Effects of Moisture Variation.

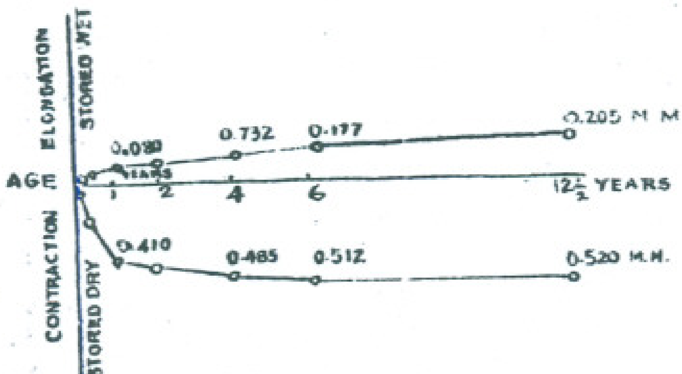
(*i*) *Linear Change in Dimensions of Plain Concrete and Stress Produced.*—Cement concrete expands in volume if immersed in water, and contracts if exposed in air. Exhaustive tests by White & Graf show that this property of concrete is not confined to the early hardening stage. Stored in air, a progressive diminution in volume is recorded and in water, a progressive increase in volume.

Very recently, some valuable research has been made by the Department of Building Research, London. The methods employed by the experts of this body eliminated errors due to thermal expansion resulting from the heat evolved during the setting of cement. The values[4] suggested on account of normal shrinkage for normal Portland Cement by the Department are :—

Shrinkage movement per unit length.

7 days	·000,02	28 days	·000,09	3 months	·000,20
6 months	·000,25	12 months	·000,32	18 months	·000,37

These values are a little lower than those arrived at by Graf, which, however, are of special interest as they cover experiments over a much longer period. Graf's values are shown in the following graph :—



LENGTH OF TEST PIECE: I METER

It may be safe to assume that the maximum expansion due to moisture content, in normal cases, will not be more than 0·000,2, while the maximum contraction due to shrinkage or drying up will be about 0·000,5.

With $E_c = 2{,}000{,}000$, a shrinkage of 0·000,5 will induce a tensile stress in concrete equal to 1000 lbs. per square inch. It is obvious that this is sufficient to crack the best concrete.

(*ii*) *Linear Change in Dimensions of Reinforced Concrete and Stresses Induced in Concrete and Steel by Shrinkage.*—When the concrete in a reinforced concrete member shrinks, compressive stress is set up in the steel and tensile stress in the concrete. Assuming that no slip takes place, *the shortening per unit length of the steel equals the net shortening per unit length of the concrete, the latter being made up of the shrinkage of plain concrete less the elongation due to tension.*

Following the notation of Taylor & Thompson, let

f_c = Compressive unit stress in concrete.

f_t = Tensile unit stress in concrete.

f_s = Tensile unit stress in steel.

$f_{s'}$ = Compressive unit stress in steel.

E_c & E_s = Moduli of elasticity of concrete and steel.

$n = E_s / E_c$ and $p = A_s / A_c$.

S=the shrinkage of plain concrete per unit.

A_c and A_s = Areas of concrete and steel.

Thus we have $f_{s'} / E_s = S - f_t / E_c$.

Also for equilibrium $f_t = p f_{s'}$.

When p is known, the value of tensile unit stress in concrete is given by the following relation, derived from the above equations :—

$$f_t = E_c . S \cdot np/(1+np).$$

Taking E_c =2,000,000, S=0·000,5 and n=15, the following values are deduced for f_t and $f_{s'}$ and net shrinkage of R. C. member for various percentages of steel :—

p	f_t	$f_{s'}$	Net shrinkage.
1 %	131	13,100	·000,435
2 %	232	11,600	·000,384
3 %	310	10,333	·000,345
4 %	375	9,375	·000,313
5 %	428	8,560	·000,286

Thus while greater amount of steel reduces the net shrinkage, it induces greater tensile stress in concrete.—It is a very interesting deduction. As soon as tensile stress exceeds the strength of concrete, cracks will take place, inspite of the heaviest reinforcement that may be put in.

5. Combined Effects of Temperature & Moisture Variation.

(*i*) *On Plain Concrete.*—A plain concrete unit having a tensile strength of 140 lbs. per square inch will crack, as soon as a contraction of 140/2,000,000=0·000,07 per unit takes place in its length. This contraction may be caused by a drop in temperature or by shrinkage or by both. It has already been shown that a temperature drop of eleven degrees Fahrenheit will cause contraction to this extent. Also that shrinkage at the age of twenty-eight days may be slightly more than 0·000,07. Any further shrinkage or drop in temperature will only increase the width of the crack.

(*ii*) *On Reinforced Concrete.*—The behaviour of reinforced concrete unit will depend on the percentage of steel. The maximum net contraction that can be sustained without any cracks will be the same as for plain concrete. The effect of reinforcement is to reduce this net contraction to a certain extent.

Reinforcement steel will prevent temperature cracks entirely so long as it is not stressed to more than n times the ultimate stress of concrete ; it cannot be stressed to its full permissible strength without cracking concrete at places.

It has already been shown that no amount of reinforcement steel will prevent shrinkage cracks, unless the concrete member is free to move.

A detailed treatment of the minimum amount of reinforcement required to keep cracks, reasonably small, is outside the scope of this paper. The worst combination is when temperature drop takes place simultaneously with swelling.

If S_c be taken to denote the tensile strength of concrete and S_e the elastic limit of steel, the minimum[10] reinforcement is given by the relation

$$\min p = S_c \;/\; (S_e - w\,E_s - n\,S_c)$$

where w is the coefficient of swelling.

Taking $S_e = 40000$ lbs. for medium steel

$S_c = 140$ lbs.

$n = 15$

& $w = \cdot 0002$

min p works out equal to 0·44 per cent.

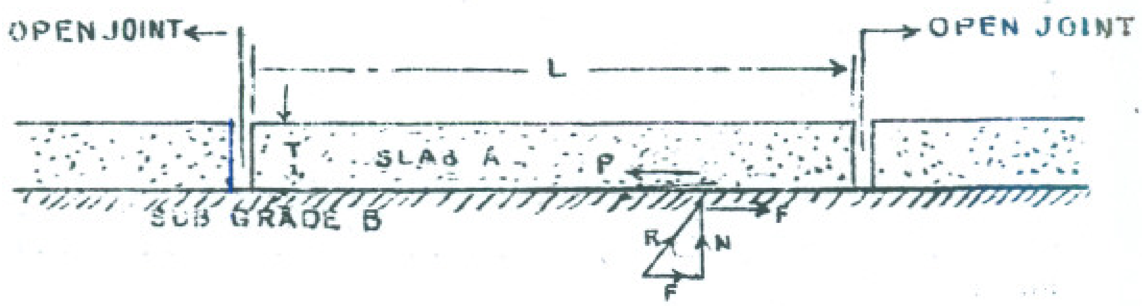
With a concrete of 300 lbs. tensile strength, the min p required comes to 1·02 per cent.

Such a high percentage of steel will be considered, in most cases, as too expensive.

It is suggested that only a minimum percentage of steel which will keep the unit tensile stress of concrete, due to shrinkage, within safe limits, may be used to take care of concrete in its early hardening stage and the effects of temperature and moisture variation should, in the main, be met by a provision of suitably designed contraction joints.

6. Part Played by Friction.

Before tackling the questions regarding spacing and type of joints arising from the above discussion of the nature of stresses, set up by temperature and moisture variation, it is desirable to have a clear idea of the part played by friction. It has been clearly stated in paras. 3 and 4 that stresses in concrete or reinforcement are produced only when the movement of the structure is checked.



Let A be one foot wide concrete slab of length L and thickness T in feet, supported on subgrade B. The maximum stress caused by subgrade friction would be half way between the two free ends.

Due to a drop in the temperature of the slab, a force equal to P is produced tending to contract half the length of the slab towards the centre. R is the reaction of B on A; the component F of R along the surface of contact is the friction and the component N perpendicular to the surface is the reaction to the normal pressure. So long as there is no slipping, F equals P. The greatest value of F obtains when slipping impends. This limiting friction is equal to $f \times N$ where f is the coefficient of static friction. This coefficient depends on the nature of the materials in contact, character of rubbing surfaces and kind of lubricant, if any, used and is independent of the intensity of normal pressure.

Goldbeck[6] has shown that the coefficient of subgrade friction in the case of concrete slab pavements may readily attain values of 1·5 to 2·0.

Assuming the weight of a cubic foot of concrete as 144 pounds, N or normal pressure for half the length of the slab would be $144 \times \frac{L}{2} \times T$ pounds. Taking the coefficient of subgrade friction as 2, the limiting value of F comes to $144 \times L \times T$ pounds. The area of the section of the slab being 144 T square inches, the stress caused in concrete, half way between slab ends would be $\frac{144 \times L \times T}{144\ T}$ or L pounds per square inch, provided the temperature change is great enough. At any other point of the slab, this stress will be equal to twice the distance in feet of this point from the nearest free end. Obviously the maximum stress in a concrete slab due to friction will occur half way between the slab ends.

It has previously been stated that one degree change of temperature is capable of producing a force of 13 lbs. per square inch in 1 : 2 : 4 concrete. It follows that a change of only $\frac{1}{13}$ degree Fah. is required to produce a stress in tension or compression sufficient to overcome the subgrade friction of a concrete slab of one foot length.

Or, that a seasonal maximum variation of 135 degrees would be sufficient to produce a stress of 1755 pounds per square inch in a slab 1755 feet long.

Also it would appear that if open joints existed at 140 feet intervals in a concrete slab of tensile strength of 140 lbs. per square inch, no cracks would take place. This, however, is not so, as there are some other important factors involved.

7. Other Factors.

(*i*) *Uneven Distribution of Stress.*—In the above discussion, only the resultant stress, evenly distributed, has been considered. The maximum temperature stress over a part of the cross section may be very much greater. Changes in the concrete temperature are brought about by the warming or the cooling of the exposed top surface. In the Arrowrock dam, with a seasonal change of 75° Fah., the variation in the concrete temperature at $3\frac{1}{2}$ feet, 10 feet and 20 feet from the nearest face was 32°, 12° and 0° Fah. respectively. According to "Highway Research in Illinois Transactions" (Vol. 87, p. 1194), there may be a difference of from 10° to 20° between the temperature of the top and bottom surfaces of a 7" concrete pavement causing unequal distribution of the resulting stress.

(*ii*) *Uneven Subgrade Support.*—The increase in resistance due to uneven subgrade support cannot be theoretically worked out, as it will vary with the degree of unevenness and the material of subgrade support. When provision for movements of the structure has been made, this factor should, as far as possible, be eliminated, by making the support level and smooth.

(*iii*) *Resistance of the Expansion Joint Filler to Closing.*—The unit resistance of the expansion joint filler also will vary greatly with the nature of the material. This will be discussed later under the subhead "Joint Filler." It may be stated here that some of the ready made joints sold in the market offer high resistance. Usually it may be assumed that this resistance will be equal to the frictional resistance. This resistance will come into play only when the slab is expanding and exerting compressive stress in concrete.

(*iv*) *Combined Bending and Temperature Stresses.*—If temperature stresses have not been allowed for in the design, these stresses, added to the bending stresses due to external loads, may easily exceed the endurance fatigue limit or even the ultimate strength of the concrete. Thus the temperature stress must be considered in conjunction with the bending stresses to serve as a guide for locating joints.

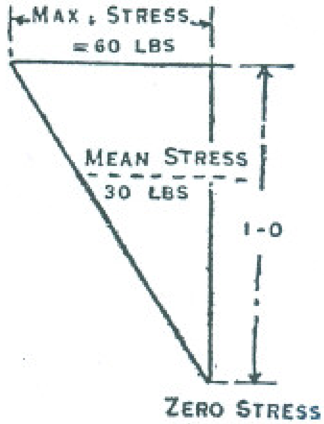
8. Width of Expansion Joints.

It was shown in para. 3, that a net effective expansion space aggregating to about 1 inch per 100 feet corresponds approximately to the expansion, expected from the minimum winter to maximum summer temperature range. As concrete is not usually placed at the lowest or the highest temperature, 1 inch per 100 ft. would, perhaps, suffice both for temperature and moisture variation. This is well borne out by general experience.

This space should be available for the expansion of the concrete structure—the minimum space occupied by the joint filler in a compressed state being extra to this. Assuming a deflection of 50% of the original thickness in the case of premoulded fillers, *a minimum joint space of 2 inches is required per hundred feet length. If the expansion joints are spaced 25 feet apart, the minimum width of the joint ought to be half an inch.*

9. Spacing of Expansion Joints.

(*i*) *Structures in Contact with Subgrade Support throughout their Length.*—Most of the authorities suggest that the spacing of joints in such structures should be from 20 to 40 ft. Let us theoretically analyse the stress, caused during contraction, in a floor slab, 1 foot thick, with joints spaced 30 feet apart. Frictional force of subgrade is 30 pounds per square inch as shown in para. 6. This is brought into action by a small drop of temperature equal to $\frac{30}{13}$ or 2·3° Fah. It is very probable that the bottom surface may not have been at all affected. The diagram of distribution of stress across the section will, in that case, be a triangle, the maximum stress being equal to double the stress caused by even distribution. Thus the maximum tensile stress in the top concrete will be 60 pounds per square inch.



To this must be added a certain stress caused by uneven subgrade and bending due to external loads and some factor of uncertainty.

The tensile strength of an ordinary 1 : 2 : 4 concrete is 140 lbs. per square inch at the age of 28 days. At the age of 2 days, the strength is only 25 per cent. and at the age of 7 days only 50 per cent. of the strength it attains after 28 days. Thus at the age of 2 days, tensile strength of this concrete will be only about 35 pounds. If sufficient care is not taken in curing concrete at this age, by covering the concrete by damp straw etc. to prevent heavy variation of temperature and shrinkage by drying up, it is bound to develop cracks, unless shrinkage and temperature reinforcement is provided. This shows that the spacing of joints is influenced by the probable care that would be taken in the curing of concrete in its first month.

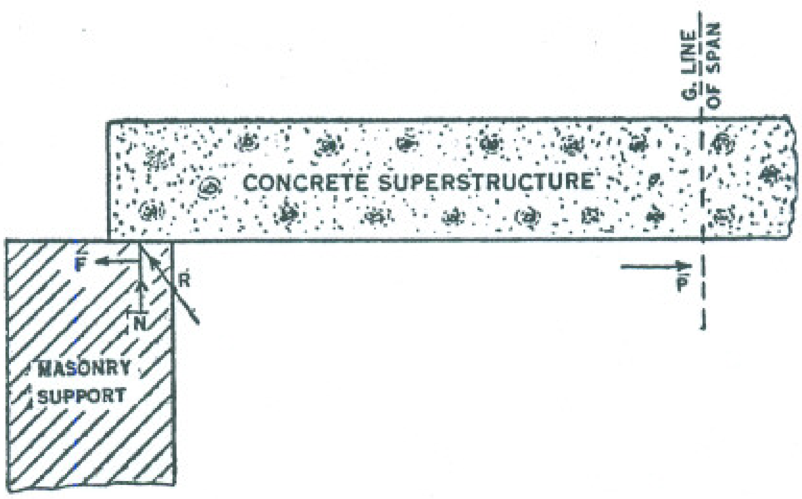
In case of thick sections of walls such as dams, joints are usually spaced further apart decreasing the spacing with the thickness of the wall. For South African conditions, Kanthack[7] suggests that the contraction joints in large dams should be provided at intervals of not greater than fifty or sixty feet. In the Pardee Dam[8], (a description of which appeared in *Engineering News Record*, February 14 and March

21, 1929) up to a height of 179 feet, (total height=359 feet) contraction joints were spaced at 150 feet intervals; from 179 feet to a height of 279 feet above the bottom, 75 feet spacing was provided and from this level to the top the spacing was $37\frac{1}{2}$ feet.

On the Sutlej Valley Weirs, practice varied with the experience gained. On the Suleimanke Headworks, which was the first to experience the trouble, due to temperature and shrinkage cracks in the completed monolithic concrete floor of the left undersluices, joints were spaced at 30 to 40 feet intervals. At Ferozepore Weir, the slabs bounded by contraction joints are something like 32 to 35 feet$\times$32 to 34 feet. At the Panjnad Weir, the last of the series on the Sutlej Valley Project, the size of such slabs, upstream of the line of gates, is of the order of 31 to 34 feet$\times$22 to 23 feet (these would always remain under water after construction) and is 22$\times$23 feet downstream of the gates.

Expansion joints, as located at Panjnad Weir, are shown in Plate 1. It is an important constructional detail that the day's work in concreting should end on all sides by an expansion joint and that there should be no horizontal layering of concrete, as due to weak bond between two days' work, this remains a weak plane likely to develop a temperature crack. At Panjnad Headworks concrete over 6000 c. ft., was placed in one block in a day, sometimes with the aid of two concrete mixers (Photo No. 1).

(*ii*) *Structures Resting on their Supports at Ends only.*—These structures deserve a special mention, as such cases are frequent. In these cases N or the normal pressure is the load transmitted by the span to one support, F is equal to $f\times N$, where f may have values from 0·6 to 0·7 for dry masonry supports[14].



Further, if a roller or rocker bearing is placed under one end of the superstructure, the resisting friction can be reduced to one half or even one-fourth. The spacing of joints may, therefore, in such cases be 60 feet or even more.

soft enough at high temperatures so as not to offer undue resistance. Yet it must not melt even at the highest atmospheric temperature, so that it does not flow away from points situated at a higher level.

At low temperatures too it must be soft enough, so that the material, forced out by previous expansion, does flow back during contraction and must be ductile enough so that it will not crack and leave voids.

Besides these, ease of installation, minimum loss in handling and a fair resistance to traffic are other desirable qualities.

It must be realized that one type of filler cannot be found to meet all these points and a selection has to be made to suit the particular requirements of each case, as best as is possible, in our present knowledge.

12. Materials for Joint Fillers.

(*i*) *Liquid Fillers.*—Asphalt is found to meet the essential requirements for a joint filler. There are, however, limitless variations in the properties of asphalt. For example, an asphalt meant for water-proofing foundation walls underground or one intended to cover a roof is not at all suited for use in expansion joints.

Specifications for a suitable asphalt must vary with the geographical position of the region and the nature of the service required in the expansion joints of the structure, and therefore standard limits cannot be fixed for all climes. A tentative specification suitable for India has been suggested by the Author in Appendix B.

At Ferozepore and Panjnad Headworks, Socony asphalt grade 101, having penetration 31—40 and melting point 132° —145° Fah. was used for filling the joint below the top 9 inches. The top 9 inches was filled with asphalt cement 115 grade, having penetration of 30-40 and melting point 160°—180° Fah. Grade 118 was found to be unsuitable on account of its very low penetration and ductility.

Pure asphalt of the right grade is perhaps the best filler. In situations where heating is inconvenient or hot liquid cannot be poured into the joint some sort of substitute, e.g., Servicised Plastic Cement, is used. Where the asphalt is considered unsightly, some patent grey filler may be used.

At Ferozepore and Panjnad Headworks, the filler was an admixture of 50% asphalt and 50% sand except in the top 9 inches of the joint, for which a stiffer mix of 60% sand and 40% asphalt was used.

In the preparation of the mastic, both sand and asphalt were heated separately to about 350° Fah. No portion of either ingredient was allowed to be heated above 400° Fah. The heating of asphalt is of considerable importance, since overheating is injurious and underheating makes the application difficult and unsatisfactory. The correct

Each case should be considered separately in the light of above remarks and allowance made in the working stresses of longitudinal reinforcement where necessary. [(Para. 3 (*iii*)].

10. Is Expansion Less Injurious Than Contraction?

At first, it would appear that, as expansion produces compression, it is easily taken care of by the strength and elasticity of concrete and that it is only contraction which causes cracks on account of concrete being weak in tension.

What happens is this. Initially contraction causes cracks. Cracks during the period of contraction go on accumulating dirt. Dirt confined between parallel surfaces is subsequently compressed by the expanding concrete. In the course of a few years, a stage is reached when the cracks are filled with hard incompressible material, which offers no possibility of relief to the forces of expansion. At this stage, internal compressive stresses are caused. Dew, springs and rains may increase the moisture content and thereby add as great expansion as caused by the maximum range of temperature. This may be made worse by unequal distribution of stress, across the section, on account of warping.

Thus stresses even higher than 2000 lbs. per square inch may result, and crush the best concrete. There can be little doubt that this is the reason why concrete roads, without expansion joints, having successfully passed a number of years, show signs of blow-ups and spalling at cracks.

Expansion also causes serious trouble at sharp bends or angles, where the lengthening of the surface may produce a sliding of one portion of the structure past the end of the other. In such cases, the forces of expansion are acting against each other. The evidences of this civil war exist in the shape of numerous cracks at the corners of concrete slabs and the right angle connections of wing walls with abutments, where such wing walls have been built at the same time as abutments, without a sliding joint.

11. Essential Properties of the Joint Filler.

From the above discussion, it is clearly seen why a successful filler for expansion joints must possess the following essential properties :—

(*a*) It should make a firm bond to the concrete surface, so that it does not pull away from the concrete during contraction and leave voids to get filled with dirt or water, a bond to render the joint water and frost proof.

(*b*) It must be insoluble in water.

(*c*) The material should be capable of being compressed within its own volume to a certain extent, when demanded by expansion without offering great resistance. To achieve this end, it should be soft enough at high temperatures so as not to offer undue resistance. Yet it must not melt even at the highest atmospheric temperature, so that it does not flow away from points situated at a higher level.

At low temperatures too it must be soft enough, so that the material, forced out by previous expansion, does flow back during contraction and must be ductile enough so that it will not crack and leave voids.

Besides these, ease of installation, minimum loss in handling and a fair resistance to traffic are other desirable qualities.

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In the preparation of the mastic, both sand and asphalt were heated separately to about 350° Fah. No portion of either ingredient was allowed to be heated above 400° Fah. The heating of asphalt is of considerable importance, since overheating is injurious and underheating makes the application difficult and unsatisfactory. The correct temperature was determined with the aid of thermometers reading to 500° Fah. The desired proportions were then placed in a mixing vat, with heat underneath and were thus thoroughly mixed together (Photo. No. 2).

To make the mastic adhere properly to the sides of the joints, it is very essential that joints should be thoroughly clean and dry. The use of an air blast from an air compressor was found very effective at Panjnad Headworks, both for cleaning and drying deep joints (Photo. No. 3).

From the mixing vat the mastic was carried, in buckets lined with wood, to the joints and after stirring it again to mix both the ingredients, the mastic was poured into the joint quickly (Photo. No. 4).

From the detailed discussion in paras. 10 and 11 it would be easy to see that *addition of sand in asphalt, used as filler for contraction joints, is not a sound practice*. During expansion of the slab, it is mostly asphalt that is forced out and when part of this finds its way back into the joint on contraction, in all probability, it carries some dirt along with it. This dirt, added to the already high proportion of the sand deposited in the joint, can offer no relief to the forces of expansion.

Here another important point arises. Even if the joint is filled with pure asphalt, by the process indicated above, it is only a matter of time for the joint to get filled up by a soil filler after forcing out all asphalt. By calculating the cumulative displacement due to daily and seasonal temperature variation, it can be easily shown that the cost of keeping bitumen joints even approximately filled up with pure bitumen at all times is too high for practical considerations.

There is a practical difficulty as well to be considered. Those who have been in charge of bitumen pouring operations know how difficult it is with the average labour to keep inert material such as, small fragments of aggregate, from getting into the joints. A few such pieces will easily defeat the purpose and once covered with bitumen there is little chance of their detection.

The introduction, in recent years, of premoulded fillers for contraction joints is a move in the right direction.

(*ii*) *Premoulded Expansion Joint Fillers*[13]. Certain patent premoulded fillers are manufactured in India and other countries. These fillers have the great advantage of being easily handled and installed, requiring no heating plant, causing no interruption in the work and reducing the waste through handling to the very minimum.

In the manufacture of these fillers, usually three materials are employed—asphalt, felt and a vegetable fibre. Of the three ingredients asphalt is the most important, so far as the function of these fillers for expansion and contraction joints is concerned.

Felt not only reduces the cost but increases stiffness and structural strength, without sacrificing plasticity, thereby enabling the joints to resist wear and penetration by pebbles, which tend to replace the bituminous material, through the action of traffic.

Felt is made from rags of cotton with a small percentage of paper.

The vegetable fibrous material, about ten per cent. by weight, is thoroughly mixed with asphalt. This fibre acts as reinforcement and serves to knit the mass of asphalt, firmly together, giving it greater qualities of cohesion. Fibre, together with a felt jacket, gives toughness and aids the joint to withstand high temperature without running, melting or sticking together.

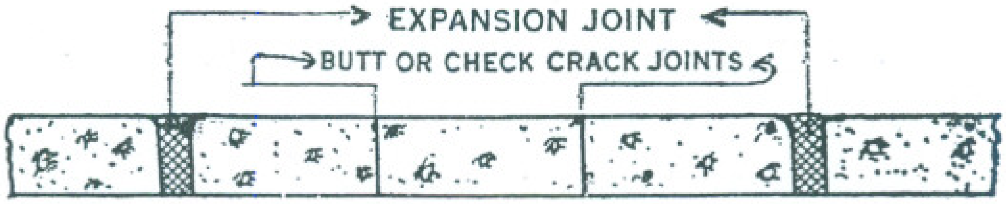
Care is needed in manufacturing these joints, so that they do not offer great resistance to an expanding slab ; at the same time they must possess a high percentage of recovery after release of pressure. This property has recently been emphasised to such an extent, as to warrant the addition of sufficient rubber into the bituminous felt filler to impart real elasticity.

These premoulded fillers are available in various thicknesses from ¼" to 1" and in varying widths and lengths. Most of the manufacturers claim a deflection of about 50% of the original thickness and a recovery (unaided) of about 60% of deflection for the better types of fillers.

13. Faulty Joints Criticised.

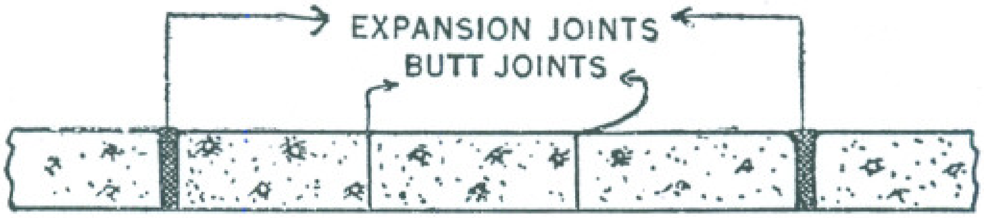
" Check Cracks " are meant to induce the crack to occur along a straight line by providing a plane of weakness. One day's work may have a butt end and the work on the following day is started straight off from this face. Another device is to put alternate slabs first, followed by intermediate ones. By this method, no actual expansion joints are provided.

Sometimes to facilitate continuous construction these so-called 'check cracks' are used in conjunction with expansion joints. Unless these are properly sealed or bonded together, nightly and seasonal contraction encourages a fairly rapid filling of the open cracks with incompressible soil. On re-expansion, cracks wholly or partially filled with soil cannot again completely close and the adjacent sections will be pushed towards the nearest expansion joints. This action, repeated many times, completely closes the expansion joints and defeats their purpose of relieving the concrete from excessive compression stresses when the concrete slab expands. What is likely to happen is made clear by the following sketches [6] :—



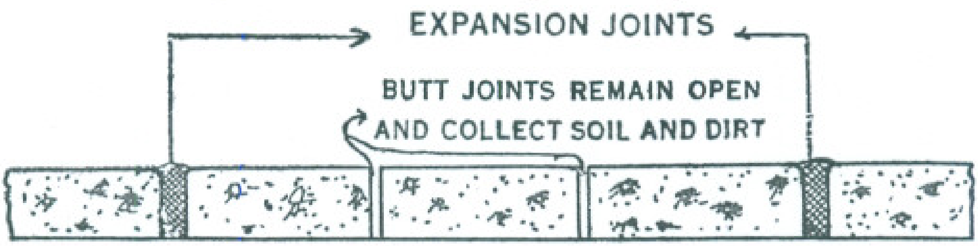
As constructed.

Expansion joints open, all other joints and cracks practically closed.



Seasonal Expansion.

End slabs pushed towards expansion joints.



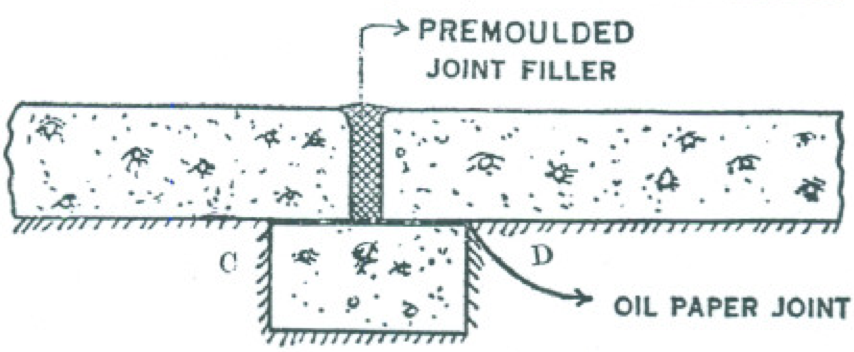
After Contraction.

All check cracks open.

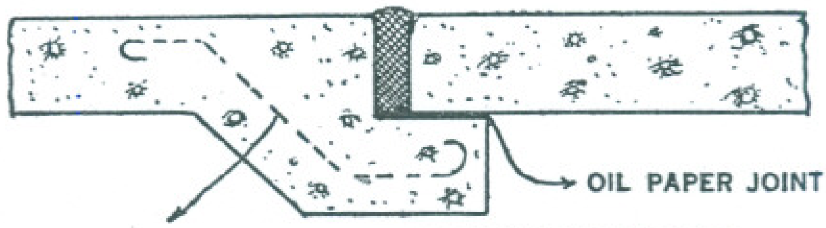
The same argument may be advanced against grouting of joints after letting the concrete stand for a season or against the simple tongued and grooved joints. For short lengths with ends unrestricted, this may seem all right, but for long structures these joints cannot be relied upon to prevent blowing up or crushing at the edges.

14. Joints Recommended for Use.

(*i*) *Joints on soft ground.*—Two designs suggest themselves. One is to place under the joint a concrete beam or curtain wall to give transverse strength at this weak point, as shown in the sketch :



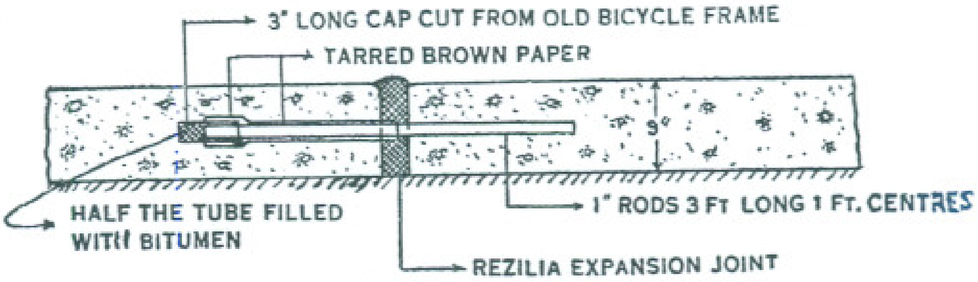
If the concrete of this curtain wall is in any way bonded to the slab concrete, it will increase the stress caused by subgrade friction due to anchorage and thus increase the chances of intermediate cracking. The plane C D should form an oil paper joint. In certain cases where there is fear of the comparatively light curtain wall shifting from its position due to heavy undersurface movements of water or due to creep under fast traffic it may be desirable to fix the position of this curtain wall, with reference to the slab ends it supports. It is suggested that in such cases one end may be tied properly to the curtain wall, while the other end is left free to move as illustrated by the sketch. In this case as the slab is anchored at one end, subgrade friction on the whole



length, and not on half the length of the slab, will determine the spacing of the joints.

The other method is to dowel the adjacent ends by means of short pieces of rails or bars. Half the length of the rail is covered with paper or coated with soap or grease to prevent adhesion to concrete. It should be realized that simple dowelling cannot make a sound contraction joint. The method[12] adopted by the County Surveyor of Kildare may be given as a good illustration.

At the expansion joints, 1 inch steel rods 3 feet long were laid at 1 foot centres at the neutral axis of the 9″ thick slab. The bars were first wrapped in tarred brown paper for half the length, and a cap about 3 inches long cut from old bicycle frames and half filled with bitumen was inserted under the paper, at the end, as shown in the sketch.



A. DOWEL SLIDING JOINT
AS USED BY COUNTY SURVEYOR OF KILDARE

The object of the cap was to prevent the fresh concrete butting up against the end of the bar. The bitumen was stiff enough to prevent the cap being forced right over the bar when the concrete was laid at the joint. Calendar's 'Rezilia' precast filler was used and this had holes cut in it, at the correct level and distance for the bars, which were pushed through the holes, until the paper was up against the filler. The wooden shuttering also had holes cut in it, at the correct intervals to allow the bars to project through.

(*ii*) *Joints for Roadway.*—Where the material forming the wearing coat is fairly elastic, the method shown in the sketch below may be used with advantage.

Types suitable for concrete wearing surfaces are shown in the other two sketches :—

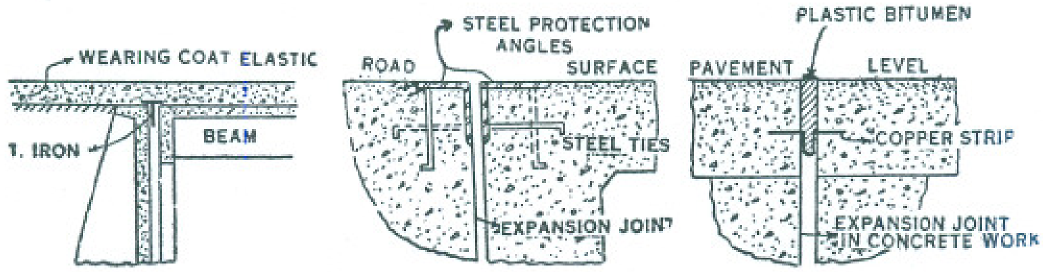


Figure 1, on Plate II shows a sliding roadway joint as used in the case of concrete road bridge at Panjnad Head Regulator. The sketch is self-explanatory. To the underside of the projecting portion of the slab on one span is fixed a steel plate, which is meant to slide on the end portion of the adjacent concrete slab without chipping off the concrete. The space between the wearing coats is filled with a bitumastic filler.

(*iii*) *Joints for Hydraulic Structures.*—A series of experiments on a bitumen joint 1 inch wide between concrete faces 2 feet deep and 3 feet long, were carried out at Ferozepore Headworks. A joint properly cleaned, dried and then filled with 18 inches of soft grade mastic topped with 6 inches of hard grade mastic withstood a pressure due to a head of 7 feet of water for 24 hours without any leakage. Further, under a head of 9 to 11 feet the joint did not show any signs of blowing, though a small amount of leakage occurred. (A summary of these experiments is given in Appendix C.)

In the case of hydraulic structures where blowing up is dangerous, some mechanical device should be provided to ensure watertightness. At Ferozepore and Panjnad Weirs, strips of malthoid 9″—18″ wide were placed in the joint, as shown in a sketch on Plate III. Malthoid, however, has short life. Copper flashing was inserted in the joints round pier wells, as at these places settlement cracks were likely to occur due to heavy live loads passing over the Railway Bridges. (See Plate III.)

A joint suitable for two adjacent reinforced concrete breast walls, as constructed at Panjnad Head Regulators is shown in Fig. 2, Plate II. Where there is no backing, as in a long retaining wall double metal stops may be used, intermediate space being filled with mastic.

15. Provision for Movements at Abutments of Girder Bridges.

In the absence of a proper provision for movement, on contraction the bridge pulls the abutment retaining wall with it. On expansion in subsequent warm weather, being unable to force the wall back owing to the earth filling behind, the deck slides over the wall and a crack is formed. A progressive action of this nature may become serious for the abutment.

In case the abutment is sufficiently rigid to prevent the above action, fracture will take place either in some of the overstressed members of the bridge or at the abutment.

It is, therefore, desirable that when the span exceeds about 30 feet, proper provision for expansion and contraction should be made at one end. A reference is invited to Para. 9 (*ii*).

(*i*) *Sliding Joint.*[11]—A common method is to form a horizontal joint entirely separating the bridge structure from the abutment wall.

Under each of the main girders, the bearings are slightly raised. Upon these, four thin rectangular plates of metal of suitable size are placed, the two outer plates being, generally, of steel about 5/16 inch thick and the two inner ones of copper about 1/8 inch thick. They need not be attached either to the beam above or the base below, or in any way to one another.

The edges of the plates should be wrapped in greased paper. When movement takes place the two upper plates slide over the two lower plates. The arrangement is shown in Fig. 1 on Plate No. IV.

(*ii*) *Reinforced Concrete Rocker Bearings.*[11]—For medium loads, reinforced concrete rocker bearings may be economically employed. The rockers are usually placed on the abutment at an inclination towards the bridge, so that the horizontal component thrust may, partially, resist the earth pressure behind the abutment. The maximum permissible angle depends upon the load carried by the rocker and the maximum calculated horizontal movements of the supported surface. The practical limit is found to be from 20 degrees to 25 degrees with the vertical. (See Figure 2 in Plate IV.)

On intermediate supports or where no help is sought in the resistance of earth pressure, these bearings are arranged vertically.

The height of the rocker is dependent on the anticipated move ment and is usually equal to the radius of curvature of the contact faces

The radius of curvature of contact surface of the rocker, when the face of the superstructure is flat, as is generally the case, is given by $r=p/k$ (from Hertz's formula[11]) where r is the required radius in inches, p is the pressure per inch of length in pounds, and k a constant depending on the material.

It is essential to use a rich mixture of concrete, not poorer than $1 : 1\frac{1}{2} : 3$ for rocker or roller bearings. The constant k for such concrete may be taken equal to 342.

The compressive stress at the actual contact area is much higher than that ordinarily allowed. This is permissible, but provision must be made to disperse the thrust immediately above and below these points and also to restrain the material against lateral expansion.

To achieve the necessary dispersion of the load, several layers of reinforcement are placed normal to the direction of the thrust. These may be in the form of gridiron stirrups.

Figure 3, Plate IV shows a typical reinforced concrete rocker bearing. These are generally precast and held in the desired position, during construction, by means of the projecting axial bars, which are partially provided for this purpose. The maximum stress upon the cross sectional area is not allowed to exceed two-thirds of the ultimate stress of the concrete used.

Care is required to ensure that all surfaces are in absolute contact and that rockers are in perfect horizontal alignment.

(*iii*) *Cast Iron or Steel Rocker and Roller Bearings.*—For heavier loads cast-iron or steel rocker bearings should be used.

A suitable bearing[3] of this type is shown in Plate V. The cast-iron rockers are placed directly under the girders in pockets in one abutment, specially left for the purpose. The rockers rest on steel plates and additional plates are placed above the rockers to receive the superstructure. The lower plates are accurately set in mortar beds, and the rockers are held by means of wooden wedges, as shown in the figure.

The pockets are filled with asphalt. A bituminous felt joint is placed along the bridge so as to prevent the slab from resting directly upon the abutments. Rectangular holes are cut in the bituminous felt to allow direct bearing of rockers, steel plate and superstructure. The top plates are held in place by means of sticks placed vertically. Stiff mortar is placed round the edges of the plates to prevent possible leakage into the pocket while the superstructure is being poured.

The radius of the cast-iron rocker face can be determined from Hertz's formula $r=p/k$ given in (*ii*) above, by taking[3] the constant k equal to 600. For cast steel, some authorities take the value of k as high as 2840.

For the heaviest loads, cast-steel roller bearings are used. These may be designed on the same lines, as the steel roller bearings for iron girder bridges of long spans.

16. Conclusions.

Cracks which to a casual observer may seem to occur at random, are caused by definite forces of nature. These forces may be set up by temperature and moisture variation besides loads.

Forces set up by temperature and moisture variation are of sufficient magnitude to crack the best concrete.

Stresses due to shrinkage present a problem which is farther from a final solution than is the problem of stresses due to loads or temperature variation. Every effort should, therefore, be made to obtain concrete with a low coefficient of shrinkage. There is still vast scope for experimentation and investigation in this direction. With the same object it is most desirable to keep a strict control of concrete during construction of any structure of importance, even to the extent of making the life of the contractor miserable.

No amount of reinforcement steel can prevent cracks entirely, if the structure is not free to move. The percentage of the minimum reinforcement required to keep cracks reasonably small is rather high and its cost may, in many cases, be prohibitive. A practical solution of the problem lies in making provision in the design to enable the structure to move freely.

In the case of long structures supported at a few points only, such provision can be made by means of suitable bearings, such as rocker or roller bearings. In the case of other structures, properly designed contraction joints provide the necessary scope for free movement.

The entire subject of contraction joints is capable of a rational treatment. The spacing and width of joints should not be considered as matters governed solely by rules of thumb.

A net space of about one inch for 100 feet is required in India for temperature and moisture movements, the minimum space occupied by the joint filler, in a compressed state, being extra. If joints are spaced 25 feet apart, the width of the joint must not be less than half an inch.

Spacing of joints in concrete structures is governed by two factors, namely, friction and the tensile strength of concrete. Reinforcement, if used to prevent shrinkage cracks forming in early stages of the concrete, will permit a bigger spacing. No general rule will cover all cases; each case should be individually dealt with. Asphalt possesses most of the qualities required in a joint filler. Asphalt of any grade will not do. It must conform to certain physical and chemical tests. A tentative specification for a suitable asphalt for India has been laid down in the paper.

It is unsound practice to mix sand with asphalt to be used as a joint filler.

A perfect joint filler has, perhaps, yet to be discovered. The introduction of premoulded joint fillers seems to be a move in the right direction. Here is an industry which can be taken up in the country with a reasonable chance of success.

APPENDIX A.

1. *Tensile Strength of Various Cement Concrete Mixes.*

Compression tests on 6" cubes of concrete picked up from the actual works (not laboratory-made concrete) executed at Panjnad Headworks, using Rohri stone ballast, local river sand and Wah cement, were systematically carried out by sending such cubes to Government Test House, Alipore, for 28 days compression tests.

Certain tests were carried out at Massachusettes Institute of Technology under the direction of Professor C. M. Spofford and Prof. H. W. Hayward (page 332 of Taylor and Thompson, third edition) to determine a relation between the compressive and tensile strengths of the same concretes. From these very ratios, tensile strengths of Panjnad concrete are worked out below:—

As per Tests at Massachusettes Institute.				For Panjnad Concrete.		
Proportions of concrete.	Tensile strength at 28 days — lbs. per square inch (*a*)	Comp-ression at 28 days. — lbs. per square inch (*b*)	Per cent. $a/b \times 100$. (*c*)	Propor-tions	Compre-ssion strength as per actual tests. — lbs. per square inch.	Tensile strength worked from relations in (*c*). — lbs. per square inch.
1 : 1 : 2	210	3,290	6.4	1 : 1 : $3\frac{1}{2}$	2,950	205
1 : $1\frac{1}{4}$: 3	175	2,500	7.0	1 : $1\frac{1}{2}$: $4\frac{1}{2}$	1,700	140
1 : 2 : 4	140	1,750	8.0	1 : 3 : 6	1,300	119
1 : $2\frac{1}{2}$: 5	110	1,190	9.2	1 : 4 : 8	1,000	92

2. *Moduli of Elasticity of Concretes of Different Proportions.*

From page 402 of Taylor and Thompson, third edition, are given below values for moduli of elasticity for ordinary concrete. Field tests made at Panjnad compare fairly with the compressive strength given for such concretes.

From Taylor and Thompson			Panjnad Tests.	
Proportion	Crushing strength at 30 days. lbs per sq. inch.	Modulus of Elasticity. lbs per sq. inch.	Proportion.	Actual crushing strength at 28 days. lbs. per sq. inch.
1 : 1½ : 3	2,300	2,500,000	1 : 1 : 3½	2,950
1 : 2 : 4	1,700	2,000,000	1 : 1½ : 4½	1,700
1 : 2½ : 5	1,500	1,800,000		
1 : 3 : 6	1,300	1,600,000	1 : 3 : 6	1,300
1 : 4 : 8	900	1,300,000	1 : 4 : 8	1,000

3. *Fall in Temperature Sufficient to Crack Various Concretes.*

Concrete mix.	Co-efficient of expansion per degree Fah. "Concrete Engineers Handbook" by Hool & Johnson, page 252.	Modulus of elasticity of concrete. lbs per square inch.	Unit stress set up per degree change of temperature. lbs. per square inch.	Tensile strength of ordinary concrete at 28 days. lbs. per square inch.	Fall in temperature sufficient to crack concrete provided its contraction is checked. Degree Fah.
1 : 1½ : 3	0·000,006,77	2,500,000	16.9	205	12.0
1 : 2 : 4	0·000,006,60	2,000,000	13.2	140	10.5
1 : 2½ : 5	0·000,005,58	1,800,000	10.0	130	13.0
1 : 3 : 6	0·000,005,37	1,600,000	8.6	119	13.8
1 : 4 : 8	0·000,005,00	1,300,000	6.5	92	14.0

APPENDIX B.

A Tentative Specification for Asphalt to be Used in Contraction Joints as a Filler, Suitable for Indian Climate.

"The asphalt shall be homogeneous and free from water, and shall not foam, when heated at 392° Fah. It shall meet the following requirements :—

(*a*) Flash point	.. not less than 392° F.
(*b*) Melting point	.. 160° —230° F.
(*c*) Penetration—	
at 32° F.	.. not less than 10
at 77° F.	.. 30—50
at 115° F.	.. not more than 110.
(*d*) Loss on evaporation at 325° F. for 5 hours	.. less than 1.0%
(*e*) Ductility	.. not less than 3.
(*f*) Total Bitumen soluble in CS_2	.. not less than 99 %.
(*g*) Bitumen soluble in CCL_4	not less than 76 %.
(*h*) Reduction in penetration at 77° F. due to heating at 325° F.	.. not more than 50 %."

APPENDIX C.

Results of Experiments on Bitumen Joints Carried Out by Mr. G. A. M. Brown, Executive Engineer, Punjab Irrigation, at Ferozepore Headworks, S. V. P. (1927).

A series of experiments on a bitumen joint between concrete faces 2 feet deep and 3 feet long, width of joint being one inch, were carried out at Ferozepore, early in 1927. A brief summary of these from a note by Mr. Brown, is given here.

(*a*) *Experiment No.* 1.—The joint was first cleaned and dried, the bottom 18 inches was then filled with soft 104 grade bitumen and the top 6 inches with hard 118 grade mastic.

The mastics were prepared in accordance with the specifications given below:—

$$\text{Mastic for lower portion} \begin{cases} 50\% \text{ 104 grade asphalt} \\ 50\% \text{ dry sand} \end{cases}$$

$$\text{Mastic for top portion} \begin{cases} 40\% \text{ 118 grade asphalt} \\ 60\% \text{ dry sand,} \end{cases}$$

all proportions being by weight.

The bottom of the joint was then subjected to the pressure of a head of water, which was increased gradually up to 5 feet and left at this head for about 24 hours. There was no sign of leakage through the joint. The head was then increased from 5 feet to 7 feet and left at 7 feet for 24 hours and again increased from 9 feet to 11 feet. A small amount of leakage was observed but no sign of blowing.

The bitumen joint could not be subjected to a higher head than this, as leakage began to occur through the cement joints. The leakage stopped, when the head was reduced to 5 feet of water.

(*b*) *Experiment No.* 2.—The joint was remade and filled as in Experiment No. 1 and was subjected to 5 feet head of water.

While still subjected to this head the bitumen mastic was removed from the top, in layers of 6 inches at a time.

There was no leakage after the top 6″ layer had been removed.

There was just a trace of leakage after the second 6″ layer was removed (one foot left in the joint).

There was a distinct flow after the third 6″ layer had been removed.

(c) *Experiment No.* 3.—In this experiment an attempt was made to produce a joint similar to what one might expect on the acutal work with careless supervision.

The joint was very cursorily cleaned and was damp in places.

The mastic filling was the same as in the above cases (*a*) and (*b*).

There was no leakage with 5 feet head of water but leakage was observed with 6 feet head.

There was no blowing. The joint could not be subjected to a higher head owing to leakage beginning at the cement joints.

APPENDIX D.

Comparative Costs of Fillers.

(*These rates pertain to the year* 1929-30.)

(*a*) Bitumen mastic (as used at Panjnad) proportion of bitumen and sand 1 to 1 by weight, 1 c. ft. of mastic weighs 80 lbs.

		Rs.
40 lbs. of 101 grade asphalt at Rs. 8 per cwt. (Panjnad Stock rate)	=	2·90
40 lbs. $=\frac{2}{5}$ c.ft. of dry sand	=	0·01
Heating ingredients, drying and cleaning joints	=	1·09
Total cost per c. ft.	=	4·0

Taking average width of joint as 1″, cost per square ft. of joint =Rs. 0-5-4.

(*b*) *Pure asphalt joint.*—

80 lbs. of asphalt at Rs. 8 per cwt.	=	5·7
Heating ingredients, drying and cleaning joints	=	1·1
Cost per c. ft.		6·8

Cost per square foot of joint=Rs. 0-9-3.

(*c*) *Rezilia expansion joint.* (Patent premoulded).—It is manufactured by Geo. M. Callender and Co., Ltd. 25, Victoria Street, London (William Jack and Co., Bombay, are agents for India).

Rezilia $\frac{1}{2}$″ thick joint may be taken as equivalent to average 1″ bitumen joint, as there is much less wastage or variation of width of joint in construction.

	Per sq. ft. Rs. *a. p.*
Cost of $\frac{1}{2}$″ thick joint f.o.r. Bombay .. .	0 9 0
Add—For Railway freight, handling and installing	0 2 6
Total	0 11 6

(*d*) *Elastic expansion joint.* (Patent premoulded).—It is manufactured by the Philip Carey Company, 90 West Street, New York.

Cost of $\frac{1}{2}$″ thick joint packed for export, alongside steamer is 14 cents per sq. ft.

Approximate net weight per sq. ft. is about 2·8 lbs. and the approximate gross weight packed is about 3·4 lbs. per sq. ft.

Freight from New York to India say \$ 9·00 a ton.

$\therefore$,, ,, ,, ,, per sq. ft. $= \dfrac{9 \times 3{\cdot}4}{2240} =$ 1·37 cents.

Clearing charges etc.	=	0·63 cents.
Total	=	16·0 cents. at Port in India.

At port in India (in Indian coin)	= Re.	0	7	6	per sq. ft.
Custom duties 15% ..	= Re.	0	1	2	,,
Railway Freight and installing at site	= Re.	0	2	6	,,
Total	= Re.	0	11	2	,,

(*e*) '*Elexoid*' *jointing*. (Patent remoulded).—It is manufactured by the Asphalt Products Co., Bombay in Parel, Bombay.

Cost of $\frac{1}{2}''$ thick joint f.o.r. Bombay	= Re.	0	6	0	per sq. ft.
Railway frieght and installing at site	= Re.	0	2	6	,,
Total	= Re.	0	8	6	,,

(*f*) *Cost of Copper Flashing Joint* ($\frac{1}{16}''$ thick sheet).—

Stock rate at Panjnad per cwt. = Rs. 77 per cwt.

100 sq. feet weighs 305 lbs.

$\therefore$ Cost per 100 sq. ft. $= \frac{305}{112} \times 77$ = Rs. 210.

Allowing for 1″ overlap in 4 ft. sheets 1·0′ wide, we have only 47″ as effective width in place of 48″.

$\therefore$ for 100 ft. join, cost of sheet $= \frac{48}{47} \times 1$ at Rs. 210 % sq. ft. = Rs. 214·5

Cost of cutting and bending to shape and placing in position = Rs. 5·5

Total cost = Rs. 220·0

$\therefore$ Cost per l. ft. $= \frac{220}{100} =$ Rs. 2·2.

REFERENCES.

1. "Concrete Engineer's Handbook," by Hool and Johnson (1918).

2. "Concrete, Plain and Reinforced," by Taylor and Thompson (Third Edition).

3. "Reinforced Concrete and Masonry Structures" by Hool and Kinne (1924).

4. Building Research Department of London Technical Paper No. 11, "Studies in Reinforced Concrete II, Shrinkage."

5. Irrigation Practice and Engineering, Vol. II, by Etchevery.

6. Articles by Mr. Calliford Older published in " Engineering News Record" dated the 3rd October 1929 and the 9th July 1931.

7. " Principles of Irrigation Engineering," by F.E. Kanthack.

8. " Article on Pardee Dam," by F. W. Hanna published in " Engineering News Record," March 21, 1929.

9. Official Record of Ferozepore, Suleimanke and Panjnad Weir Divisions, Sutlej Valley Project.

10. Proceedings of American Society of Civil Engineers, February 1932, " Stresses in Reinforced Concrete Due to Volume Changes " by C. P. Vetter.

11. " Reinforced Concrete Bridges," by W. L. Scott, 1931.

12. " Indian Concrete Journal," 1929.

13. Literature published by various Companies manufacturing premoulded joint fillers.

14. Merriman's " Handbook for Civil Engineers."

PHOTO No. 1.

PHOTO No. 2.

PHOTO No. 3.

PHOTO No. 4

PLATE I.

SPACING OF EXPANSION JOINTS AT PANJNAD WEIR

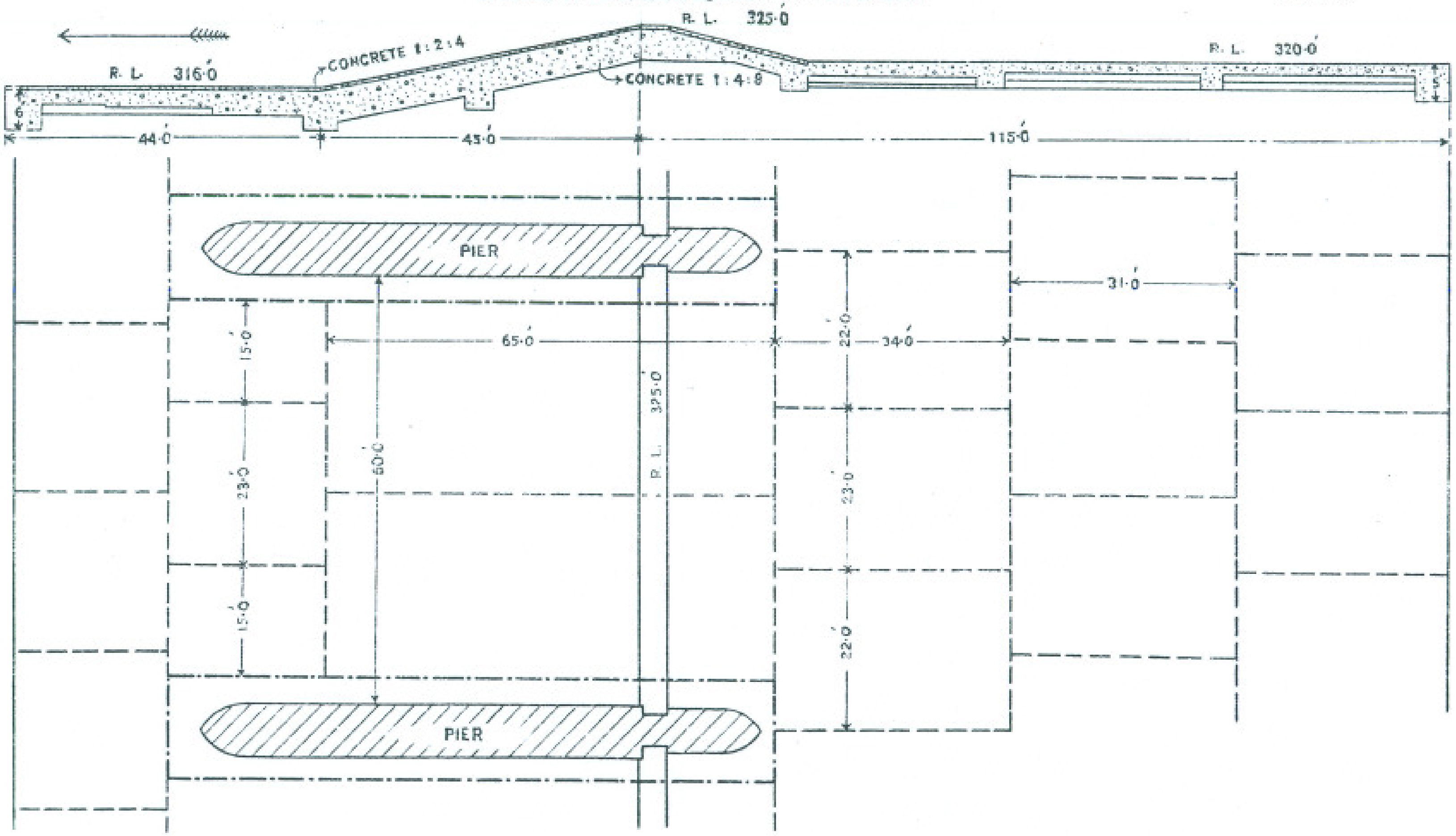
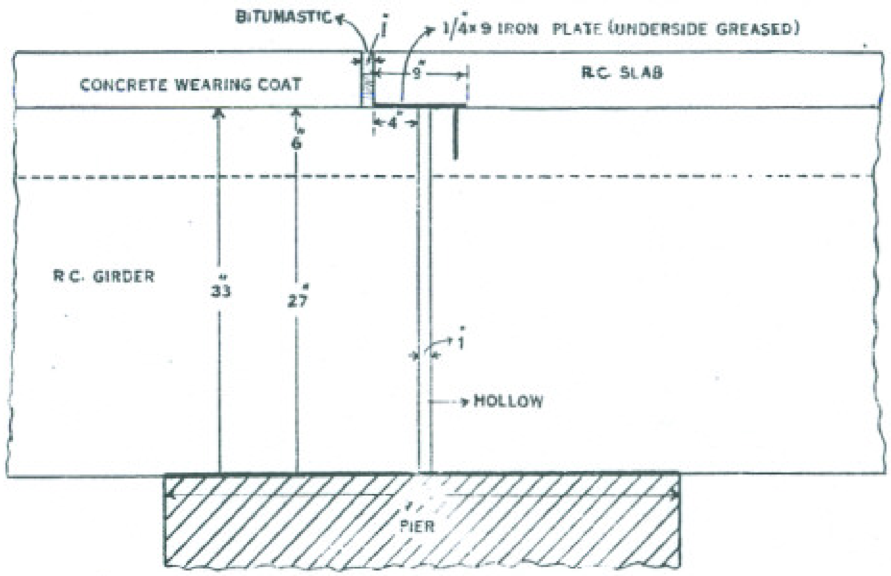


PLATE II

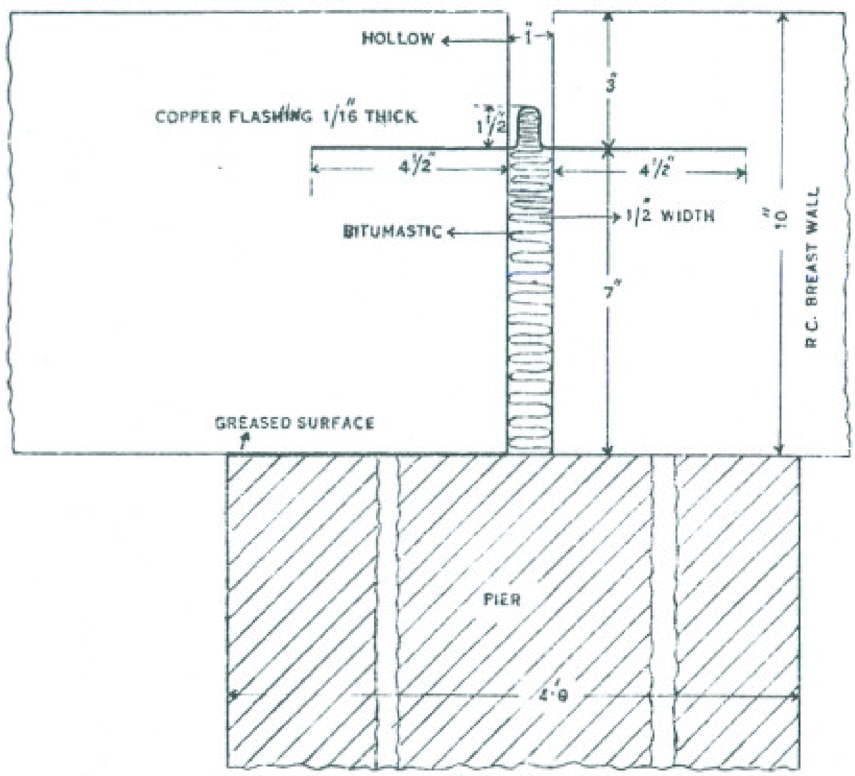
DETAILS OF EXPANSION JOINTS AS USED AT PANJNAD HEAD REGULATOR

SLIDING ROAD WAY JOINT (CLEAR SPAN 26-0)



SECTIONAL ELEVATION
FIG. 1

JOINT BETWEEN BREAST WALLS (CLEAR SPAN 26-0)



SECTIONAL PLAN
FIG. 2

PLATE III

DETAILS OF EXPANSION JOINTS IN PANJNAD WEIR FLOOR

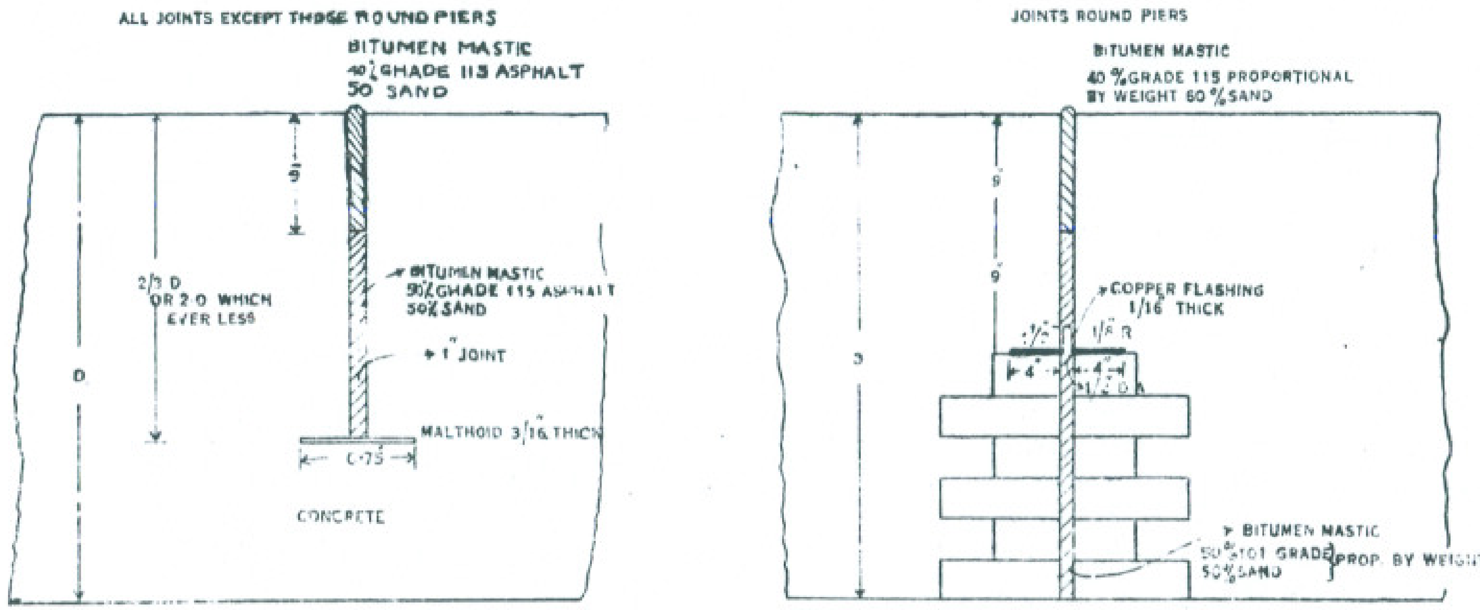
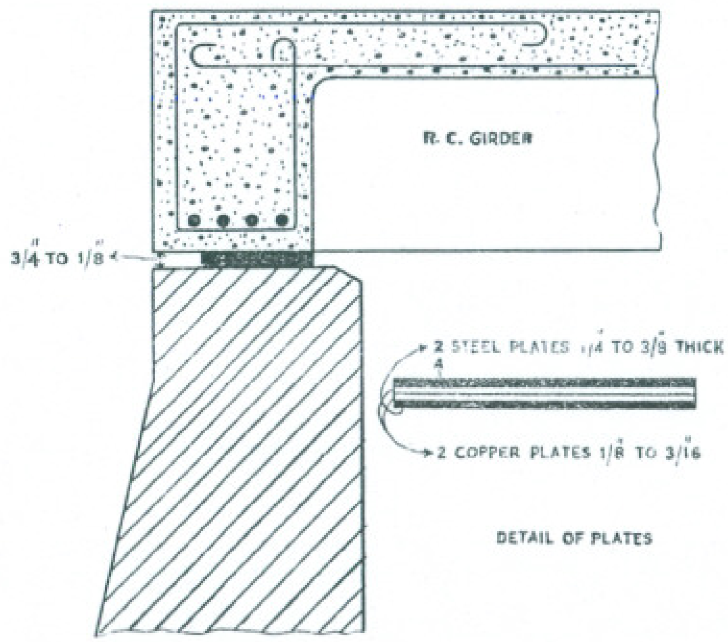


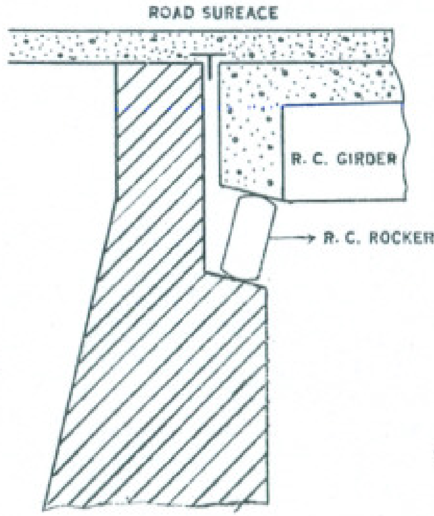
PLATE IV

PROVISION FOR MOVEMENTS AT ABUTMENTS OF GIRDER BRIDGES

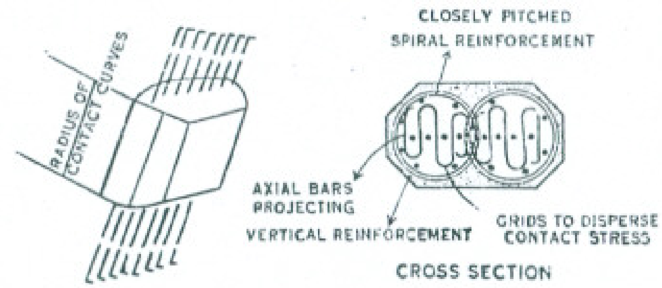
WITH ACKNOWLEDGEMENTS TO REINFORCED CONCRETE BRIDGES BY W. L. SCOTT.



SLIDING JOINT
FIG. 1.



ROCKER IN POSITION
FIG 2

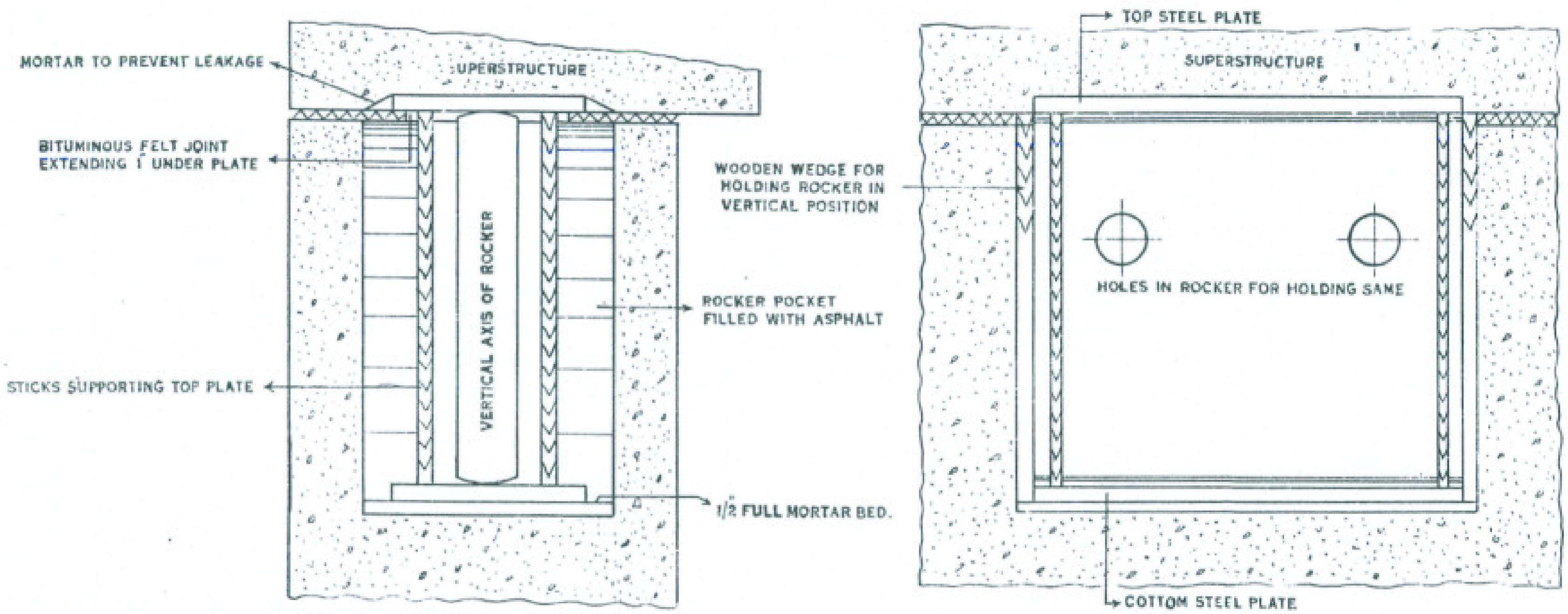


CROSS SECTION

R. C. ROCKER BEARING
FIG. 3.

PLATE V.

DESIGN OF CAST IRON OR STEEL ROCKER BEARING



WITH ACKNOWLEDGEMENTS TO REINFORCED CONCRETE AND MASONRY STRUCTURES BY HOOL AND KINNE.

DISCUSSIONS.

Introducing his paper, the **author** remarked that the coefficient of expansion and contraction due to temperature variation is practically the same for concrete and steel. It is this fact that renders reinforced concrete possible and a suitable material of construction.

Any slight difference of stress in the concrete and the steel embedded in it, due to change of temperature of these two materials, are of little practical importance, and are neglected in the design of reinforced concrete work.

Quite distinct from the above, however, are stresses which may be developed in reinforced concrete structures owing to external restraint against temperature or shrinkage movements. It is these stresses which the paper deals with.

The writer had been struck how unsightly cracks marred concrete structures. While at Panjnad this question came to mind: Was there any method to prevent cracks at least at certain places, where they may be very unsightly or dangerous? The reply came that the cracks can be localised. An expansion joint may be defined as a designed crack, affording an opportunity of using some devise to seal the crack.

Further questions came up. What should be the width and spacing and type of joints? Is contraction more dangerous than expansion? Are expansion joints more efficient than temperature reinforcement? A reference to the existing text books did not satisfy the inquisitive mind.

There is little doubt that the problem is very important. But before it can be solved it must be understood, and before it can be understood, it must be carefully studied.

Only normal portland cement concrete has been dealt with in the paper. The coefficients for rapid hardening and aluminous cement concretes are different from those stated therein. Also the discussion in the text has been confined to 1 : 2 : 4 concrete only. Information regarding other mixes is given in Appendix A.

On page 40 of the paper, there is a small misprint in the graph. At the age of 4 years, the elongation should be 0·132 mm. and not 0·732 as printed.

For another type of design of joint suitable for roadways on soft ground, a reference is invited to page 14 of the January 1935 issue of the *Indian Concrete Journal.*

Tests made on the New Jersey roads in the States have indicated two conclusions. These were:—

(1) The joint must be as nearly water-tight as is practicable.

(2) It must have ample strength to transfer full traffic load across the interruption made by the joint.

It appears that the exact alignment of dowels both horizontally and vertically is the very essence of a good joint.

Mr. G. R. Sawhney agreed with the author that reinforced concrete work does and should appeal to engineers but he would say that the backwardness in taking to that kind of work was in the past more due to lack of enterprise than the climatic extremes. The unsightly cracks appeared more to be due to lack of supervision because the fact that provision for temperature variations should be made when designing any reinforced concrete work had been known to engineers for a very long time. To guard against those cracks, as India was also a land of temperature extremes, very special care must also be taken in mixing and laying concrete as a lot more harm could be done by lack of supervision than merely not choosing the very best material as fillers.

The speaker said that the joint fillers as defined by the author were no doubt suitable for works under water but for ordinary road slab work which was allowed a wearing coat even card-board joints had proved quite successful.

Mr. N. D. Gulhati said that the author had given them some very useful information in the paper. He only wanted to touch a few points.

While, in general, it was necessary to provide expansion joints in concrete structures, there were some, in which the various members were so well bonded together that the whole structure was free to move as a unit. In such cases, for example, the walls and floors of a concrete building, no expansion joints were necessary.

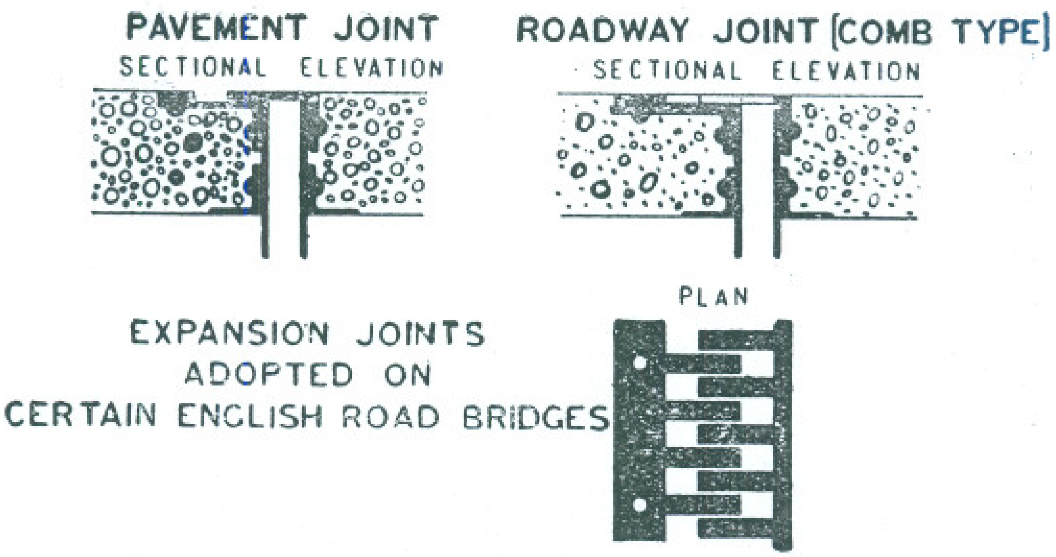
The author had not given them any advice regarding expansion joints in ordinary concrete floors in buildings. The speaker further remarked that he had seldom seen a concrete floor in which bitumen or asphalt fillers had proved satisfactory. That was probably due to two reasons. The material in itself was perhaps not suitable for the purpose, and secondly, it was not possible for small jobs to get material to any of the author's specifications. In a concrete floor inside a house, it was not so much the variation in temperature, that caused trouble, as the shrinkage that took place in the concrete when it set. The speaker had found by experience that if the joints were left alone for a period of about a month, and then filled by a lean mortar of cement and sand, it worked fairly well, provided the floor had been laid on a carefully prepared rough surface. The rough bed of the floor acted in the same way as shrinkage reinforcement and did not allow big cracks to occur.

For ordinary village road bridges, where traffic was light and up to 20′ spans a ¼″ joint left hollow right through the slab and wearing coat had been found by the speaker to do very well without any protecting angle irons. For heavier traffic and for wider joints the angle protection was of course necessary.

The speaker said that it had not been emphasised in the paper that the expansion joints in a concrete slab should not be made continuous, but should break joint. It had been noticed that if four corners of different slabs were allowed to meet at one point, there was also a certain amount of bulging up of the slabs at that junction.

Mr. P. S. A. Berridge said that he had read the author's paper and was particularly interested in the expansion joints on roadways, though his experience had been mainly on steel bridges.

He had the accompanying diagrams prepared and it showed types of expansion joints which were designed ten years ago before he came to India and were used at the ends of spans on the Scarborough Valley bridge (4 spans of 150 feet) and also on the temporary Waterloo Bridge over the Thames.



The comb type of joint was the better and was used on the roadways whereas the more simple type was put on the pavements; but both types had their disadvantages. The points where the road metal, concrete or, as in the case of the London bridges, wood block paving meets the steel angle were very vulnerable, and so far, he was not aware of any joint having been evolved which could get over that trouble.

Mr. A. N. Khosla considered Mr. Kanwar Sain's paper as a most useful contribution in so far that it dealt with a problem which was met with in every branch of Engineering. The author had focussed attention on certain fundamentals and had put together most of the available information in clear and concise form. The subject, though of such vital importance, was only vaguely understood by most members of the profession. There was a great diversity of opinion and practice regarding provision for expansion and contraction due to temperature variations etc. and a clarification of ideas in this respect would be of great practical use.

The author's observations on the functions and properties of joint fillers and suggestions regarding those most suited to different cases, were most useful. He had rightly drawn attention to the doubtful economy obtained by the addition of sand in asphalt and considered this practice as unsound. He had rightly laid stress on the thorough cleaning and drying of the joints to enable proper adhesion of the mastic to the sides of these joints. The paper was specially welcome due to fulness of details.

The author had discussed the methods of laying concrete in the floors of the four Sutlej Valley Weirs. At Ferozepur, Suleimanke and Islam, the concrete was laid in layers with varying treatment regarding expansion joints. Suleimanke had some trouble during construction when considerable areas of concrete, where faulty adhesion of layers was suspected, were removed and relaid. No further trouble had been reported from that headworks. At Ferozepur and Islam the surface layers got separated after the opening of the weir and were repaired or replaced as convenient. At Panjnad, the entire depth of floor was laid at once so that there were no horizontal planes of bedding. The work was done in compartments and the vertical joints between those were filled in subsequently.

At the Khanki Weir under reconstruction, the two methods of laying concrete, *viz.*, in layers with no expansion joints or in compartments with vertical joints, came in for a good deal of discussion. It was finally decided to lay the concrete in layers with provision for binding each layer to the layer below. The main deciding factor may be summed up as below :—

(*a*) When concrete is laid in compartments in thicknesses ranging from 3 to 6 feet, the setting heat of concrete may set up interval stresses which might cause unforeseen shrinkage cracks in the compartments when the concrete cools down. The setting temperature of concrete may be as high as 136° F. Assuming that the concrete was laid at an average temperature of 70° F., the rise in temperature due to setting would be between 60° and 70° F., a very big range. If the concrete were laid in layers, the setting heat of one layer would be mostly dissipated before the next layer was placed and the vertical hair cracks, if any, would be isolated in each layer, thus eliminating the risk of through vertical cracks.

(b) The vertical joints round each compartment would necessarily be difficult to clean, especially if they were of any considerable depth with a grid of surface reinforcement projecting from either compartment to give suitable overlap.

(c) Better quality of work can be assumed where concrete is laid in layers than where it is laid in greater depths.

These three factors are in favour of laying concrete in layers provided it is possible to tie the various layers together beyond doubt. The upper layer will expand and contract more than the one below and adjoining it due to the greater range of temperature to which the former would be exposed. If the bond is defective the layers will separate and become ineffective as a mass.

At Khanki, the concrete was laid in layers and vertical stirrups (5/8" rounds) were bedded in the bottom-most layer and projected up to the top layer so that the various layers were tied to one another by vertical reinforcement. As a further precaution long stones were bedded in each layer projecting half way into the layer above. These, besides saving in concrete afforded wide areas against shear. Thirdly, each layer was carefully nicked, cleaned and given a layer of grout immediately before the next layer was laid. Similarly all projecting stones—called binder stones were cleaned, and given a layer of cement grout before they were bedded into the layer above. The entire vertical reinforcement was effectively tied into the surface grid of 5/8" bars 9" mesh. The layer of concrete resting on the sand was 1 : 3 : 6, the top-most 1 : 2 : 4 and the intermediate layers of 1 : 4 : 8 concrete. The thickness of the layers ranged from one foot to 1·5 ft. and occasionally 2 feet.

On the Hoover dam, with a base thickness of several hundred feet, a series of pipes were inserted in the entire mass through which cold water was kept circulating throughout the period of setting. In this way most of the setting heat was abstracted and the range of temperature variation was reduced to within very narrow limits. The process was necessarily expensive but was most effective as it eliminated the possibility of unknown internal cracks due to setting heat. The seasonal variations though considerable, do not penetrate deep and therefore are relatively much less harmful.

The speaker also explained his own experience about floors and roofs. He advocated light reinforcement, say 3/16" rounds 9" in floors and 3/16" rounds 6" mesh in exposed roofs. According to his experience such floors and roofs had stood the test of at least three years without any signs of cracks or buckling. In that way, the entire floor of a room could be laid as one slab instead of in compartments which have a tendency to buckle at the corners.

Mr. G.H. Hunt heartily congratulated the author on the production of an extremely interesting paper but he would join issue with him when he said that the addition of sand in asphalt used as a joint filler was not sound practice.

The speaker said that on the Uhl Hydro-Electric Scheme a great deal of the Headworks upstream of the tunnel intake were carried out in reinforced concrete liable to considerable sudden extremes of temperature varying from a hot midday sun to sharp frost at night, and in the vertical joints on the walls at any rate, they had found a sand bitumen mixture very effective though there was a very little tendency for the filler to return back into the joint on contraction. It necessitated the occasional filling up of the joints from the top.

The reservoir with a capacity of 7 million c.ft. was some 2000 feet in length and 30 feet in depth; the bank on the river side consisted of a bund with the usual 2 to 1 slope. For water tightness lightly reinforced concrete slabs 4″ in thickness and $8' \times 4'$ in size were used originally, the size of the slabs being increased to $12' \times 6'$ during the later stages of the work.

As regards the joint filler a specification was sent to the Stores Department and four samples were received for test. It was found that the behaviour of those varied considerably, as some of them stiffened up and became much less plastic after a short time, whereas others did not appear to vary appreciably. Being apparently cheaper one of the latter was selected and that consisted of a mixture of bitumen and asbestos. To reduce the tendency to run they had the alternative of using powdered asbestos or a very fine sand. The latter was used on the ground of economy and had, he was told, proved quite successful. It must be noted, however, that the reservoir was never empty under normal working conditions though there might be an appreciable daily variation in the water surface level. Connecting the decantation chamber with that reservoir were two ducts 6 ft. and 8 ft. in diameter, of lengths of some 5000 ft. and 2000 ft., under heads of the order of 10 and 40 ft. respectively. For those the provision of expansion joints was considered prohibitively expensive. He should be very much interested to know whether the author could suggest a suitable type other than something similar to the ordinary joints used for steel pipes. They have found that notwithstanding longitudinal reinforcement and the usual precautions to ensure as good a joint as possible, fine hair cracks occurred at circumferential construction joints. The cracks were very small as the ducts were buried below ground level as soon as possible. They were treated by cutting a chase about $1\frac{1}{2}''$ deep round the circumference; lead wire of varying diameters was then driven in, the job being then plastered up to give a smooth surface. Originally the chase was fish-tailed, but that was not considered worth the increase in cost.

On test under full head the leakage was immeasurable though there were a few very small damp patches which appeared on the external surface of the concrete.

As regards roadway joints, he had recently seen one which consisted of rubber or bitumen sheeting secured by steel pins driven into the formation, the upper portion of the joint being filled with a 60 % tar and 40 % bitumen filler. That had the advantage of acting as a form for the concrete.

The edges of those joints were bull nosed with a strip of special concrete of granite chipping 6" wide and 2" thick.

Replying to Mr. Sawhney's criticism, the **author** informed him that he had very good supervision of his work.

To Mr. Sawhney's query as to which design will be more effective the author replied that in his opinion card-board would be very suitable.

In reply to Mr. Gulhati's query, Mr. Kanwar Sain informed him that when the structure is free to move, no joints are necessary. He was of the opinion that pre-moulded joints would solve the problem to a very great extent.

In reply to Mr. Berridge's remarks, the author said that he was obliged to Mr. Berridge for his introduction of a very suitable addition to the discussion by giving another type of joint used.

Replying to Mr. Khosla's query, the author informed him that he had been putting pre-moulded joints and had found them fairly satisfactory.

Replying to Mr. Hunt, the author stated that sand-bitumen was found very satisfactory. It was found necessary to refill the joints. The size of slab used was $12' \times 16'$, but as to their efficiency this will be proved in time.
