PAPER No. 85.

NOTES ON THE DESIGN OF GIRDER-BRIDGE ABUTMENTS, FOR BROAD-GAUGE (5'6") TRACK.

N. G. WATSON.

Introductory.

1. The present tendency is for engine and train loads to increase from year to year, and while this is receiving attention in the design for girders, the present official North-Western Railway Standard for these being the 1903 plus 25 per cent. there appears to be no standard method for the consideration of the strength and stability of abutments. Very often the newer and heavier girders have been and are being placed on the old masonry work.

2. A good deal of misconception also appears to exist as to how the forces act and how they should be dealt with, in considering the stability of abutments. To one who had not before to tackle problems of this nature, and when the only guides available were some obviously out of date blue-prints, designed when engine and train loads were considerably less than at the present time, it became a serious question. One officer in sending up a design based on these old blue-prints, said he was unable to satisfy himself on the stability of the abutments by any of the known methods which were accepted. A senior officer of the Railway consulted, considered it was a question for the Bridge Engineer to tackle and left it at that. Another Railway officer said that the method usually adopted was to take the vertical height from floor to formation level, to lay off an offset at floor level 1/3 of this height, the top thickness being determined by the space required for the bearing of the girders, plus the space left for expansion and the ballast wall combined. These two points were then joined up and the back of the wall stepped outwards beyond where the perpendicular met the inclined line. When further questioned about how the increased loads were to be dealt with, the reply was to add 25 per cent. to the base thickness and proceed as above described. He added that some people had not the strength of their convictions and when working on the above lines were not satisfied with the top width being the same for a considerable height (until it met the inclined line), and so commenced stepping the back face earlier than necessary.

3. There appears to be little literature on the subject, and such as there is, is either scattered or vague and indefinite. It is therefore hoped that these notes will be of some use to others, who may find themselves called upon, like the author, to deal with a subject that falls outside their usual duties, and that they will lead to the discussion of what is essentially a very difficult problem.

Loads and Forces.

We have to consider the following :—

(*a*). The dead-load, consisting of the structure itself, the girders and permanent-way and the triangle of earth between the back face of the abutment and the line representing the plane of repose. Part of this triangle up to the dotted vertical line assists stability of the structure.

(*b*). The live-load, consisting of the engine and vehicles in full running order. It would be safer when considering this to take a number of coupled engines as forming the live-load.

(*c*). Wind pressure against the side of the train and girders, and on the abutment. These forces however are slight when compared with (*a*) and (*b*) and may be neglected, except in large spans.

(*d*). Impact, and the effect of the application of brakes on or near the bridge approaches. These forces have to be considered and allowed for when dealing with the strength of girders required, but do not appear to be of much consequence when considering abutment thicknesses.

It is believed that from experiments for Impact made on different railways in India, based on the assumption that the driving wheel of an engine travelling at speed could not jump more than $\frac{3}{4}$ of an inch, the following results were obtained :—

6 inches	below	the	surface	75	per cent.	of	live-load.
1 foot	,,	,,	,,	50	per cent.	,,	,, ,,
2 feet	,,	,,	,,	16	per cent.	,,	,, ,,
3 ,,	,,	,,	,,	9	per cent.	,,	,, ,,
4 ,,	,,	,,	,,	5	per cent.	,,	,, ,,
5 ,,	,,	,,	,,	3	per cent.	,,	,, ,,

and below this it was negligible.

It will therefore be realized that this force acts chiefly against the parapet or ballast wall, and may be neglected when considering abutments over 5 feet in height.

Possible variations of the Live-load.

There are the following cases to be considered :—

(*a*). Span unloaded, with engine on the bank in such a position that the maximum load lies between the vertical line from the base of the abutment to the formation and the line of rupture.

(*b*). Span fully loaded and no load on bank.

(*c*). Both loaded.

The worst case is with the loading described under (*a*) above, which will now be dealt with in detail.

Method of Calculation.

The loads due to the engine and permanent-way, are first taken and reduced to an equivalent surcharge height of earth above the formation level. This is done by taking the weight of the permanent-way material, including ballast if any, for a standard length of 36 feet rails, and reducing it to pounds or tons per foot run. To this is added the engine weight per foot run (total weight divided by overall length) and the sum divided by the base on which this load acts. This base may be taken as a sleeper length in the case of a non-ballasted line or 10 feet, and as 14 feet in the case of a ballasted line. This gives the super-imposed load per square foot which can be reduced to the equivalent of earth giving the surcharge height h', *vide* diagram.

The weight of the girders and permanent-way carried by them are next considered the total being divided by 2 to obtain the weight to be carried by each abutment. The abutment length or face is determined by the gauge of the track, and for 5 feet 6 inches must be a minimum of 16½ feet though a dimension of 20 feet is recommended. The weight per foot run on the abutment is therefore the figure obtained divided by this width and is termed W_1. The weight of the abutment has next to be worked out, the best method is to first find some section to form a working basis. This is ordinarily a tedious process of trial and error and the following method, if adopted, will save a good deal of time.

All ordinary bridge girders are standardized and therefore for a given span the length of the bearing plate, the expansion space, and the thickness of the ballast wall combined will be known factors. Therefore for the same size of clear span the abutment thickness will be a fixed dimension at the level of the bottom of the girders. Take this width down to the floor or base vertically, add a 1/12 batter to the front face, and a 1/6 batter on the back and an economical trial section results. For those who do not care for battered faces owing to the cutting of bricks or stone involved, the remedy is to step the back face beyond the batter line in a series of vertical faces and to increase the base width to compensate for the vertical instead of a battered front face.

The empirical method described above can be tested or replaced by the following formula obtained from Rivington's Building Construction :—

$$t = Kh\sqrt{\frac{w}{w_1}}$$

where t is the mean thickness of the section, K a co-efficient, h the vertical height above the base, w the weight of a cubic foot of earth and w_1, that of a cubic foot of masonry.

The value of $K = 0{\cdot}7 \tan\left(\frac{90 - \theta}{2}\right)$. Where θ ranges between 37° to 31° for fine dry sand, K becomes 0·35 to 0·40 and hence a mean value of 0·37 may be safely assumed in all ordinary cases for K.

The weight of the section, for a strip a foot wide is worked out and termed W.

Finally we have to deal with the earth pressures ; our case will be somewhat similar to that of an ordinary retaining wall with a surcharge load.

Diagram.

The diagram given is not to scale and merely intended to illustrate what has been said above.

It gives a vertical section through the abutment and the forces are first considered with reference to the horizontal joint AB.

The single hachured area of weight W_2 acting through its centre of gravity helps towards stability, while the triangle on its right, bounded by the line of the plane of rupture and including the double hachured area of the surcharge tends to act against stability, the worst strain is thrown upon the abutment when the engine is on the track covering this area.

The angle of repose θ in ordinary cases may be taken as 33°-42′, and ψ as half that value. This assumes a natural slope of repose of $1\frac{1}{2}$ to 1.

Formulae used.

The following formulæ are suitable for the consideration of the effect of the earth pressure on the abutment wall section :—

$$P = \frac{wh}{2}(h + 2h')\frac{1 - \sin\theta}{1 + \sin\theta} \quad \ldots\ldots\ldots\ldots (1)$$

$$Y = \frac{h + 3hh'}{3(h + 2h')} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where P is the horizontal component of the total arth pressure,

w the weight of a cubic foot of earth,

h the vertical height of the abutment above the base,

h′ the surcharge height (live-load, etc., reduced to the equivalent depth of earth load),.

θ the angle of repose,

and Y the height of point of application of the force P.

Having found the value of P and its point of application, it is

necessary to determine the weights of W, W_1 and W_2 (*vide* diagram) find their respective centres of gravity, and then by taking moments about the point C or D, or by graphical methods, determine the resultant vertical force and its line of action. The force P is then combined with the frictional resistance on the plane DE to obtain $P^1 = P \sec \psi$ the total earth pressure. By producing this force until it intersects the resultant vertical force, and by plotting their values along their respective lines of action, the parallelogram of forces can be completed and its diagonal R becomes the final resultant force on the base both in magnitude and direction.

According to some authorities it is not necessary for the resultant to pass within the middle third of the base, and these authorities claim that provided it passes through the middle half of the base it is sufficient. This latter condition will however entail tension. For a discussion on this point reference may be made to Buckley's Irrigation Pocket Book and it need not therefore be repeated here.

It is however considered best to so design the structure that the resultant will pass through the middle third, but as near to its outer limit as possible, as this assures economy. Care must however be taken to see that the foundation pressures do not become unsafe. This question is referred to below.

Wing and Return Walls.

These are intended for holding up the earth of the embankment. They are constructed to suit local conditions and are built either in a line with the abutments or at an angle to them. In most cases they afford additional support to the abutments, but this should not be taken into consideration when dealing with the stability of the latter as it merely adds a factor of safety to a problem in which there are several unknowns.

Bed Blocks and Ballast Walls.

Formerly wooden bed blocks were used, but these are now out of date. A very suitable form of bed block is a cement concrete one, made in situ, the dimensions naturally varying according to the span. The North-Western Railway have a special type design for these, which gives the dimensions required for the Standard Spans adopted by them.

The design of the Ballast Wall requires some consideration. It should be strong enough to resist the earth pressure and impact. It should not be carried right up to formation level, but stopped a foot or so below and slotted to take a made up rail girder as otherwise it is sure to be damaged by the gangs when packing the line.

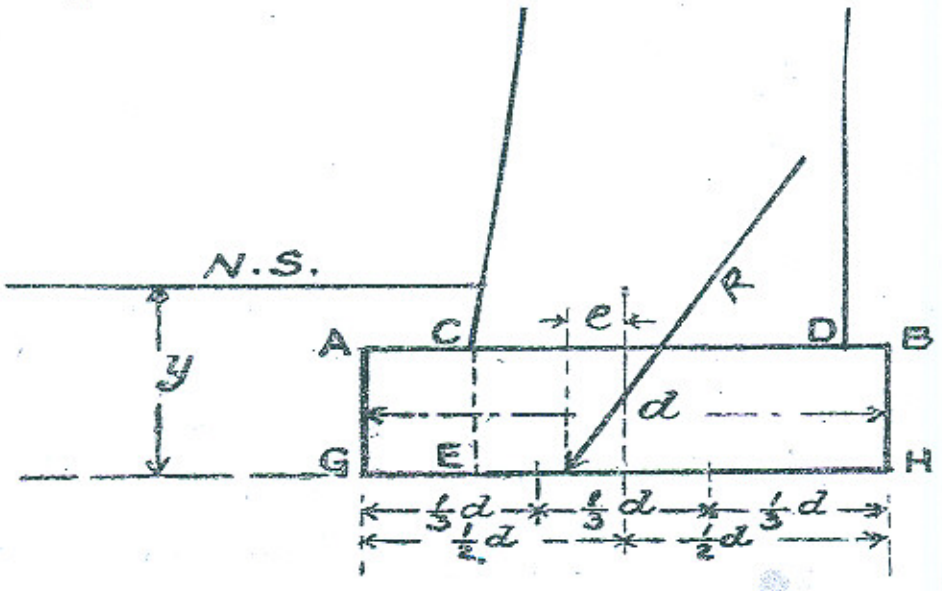
Load Diagrams for Engines.

A few of the more important classes of engines in use, are shown with dimensions in the form of load diagrams. These will be found useful in determining the possible loads that have to be provided for when considering the design of abutments for a given span. The table of approximate overall weights for various spans of Standard girders, (1903 + 25 per cent.) strength, will also be useful.

Foundations.

This subject can only be treated in general terms. Almost everything depends upon the nature of the soil and local conditions. Where scour is likely to occur it is necessary to provide either deep foundations or to protect them from scour by stone-berms or curtain walls. So far the discussion has been confined to the horizontal joint AB (*vide* diagram) but it is essential to carry the process further and to consider the effect by a similar analysis for the base GH. The line of action of the final resultant should be kept within the middle third of the base and from its position the limiting pressures at G and H calculated by the usual formula for uniformly varying stress $P = \frac{F}{d}\left(1 \pm \frac{6e}{d}\right)$ where F is the total vertical component of R, d the distance GH or width of base, and e the eccentricity of the resultant. The softer the ground the less e should be. The depth of G can be tested by Rankine's formula where wy must be $\geqq P\left(\frac{1 - \sin \phi}{1 + \sin \phi}\right)^2$ P being the pressure at G, y depth of G below natural surface, w the weight of a cubic foot of earth and ϕ the angle of repose.

The concrete base must also be sufficiently thick so that there is no danger of the offset breaking across :—



In the above diagram ACEG would be a cantilever loaded on the underside and tending to break off at EC. For testing it may in most cases be assumed that p is uniform over GE. Then deducting the weight of the block ACGE, the nett bending moment on the section CE can be calculated from which the tension at E is obtained. This tension must be kept within safe limits for the materials used. There must also be safety against the wall sliding on its concrete base, and on the plane GH. This is easily tested by comparing the total weight of the wall multiplied by the co-efficient of friction with the horizontal thrust from the backing. It will be noticed that for the plane GH the abutting power of the earth in front of the wall can be taken into account.

Conclusion.

My acknowledgments are due to the North-Western Railway Staff of the Construction Branch, for the help given and for the blue-print type designs received. It only remains to add that many abutment walls have been designed by the methods above described, for the Sutlej Valley Project Quarry Railway, they have now been tested under heavy stone traffic, and have shown no signs of weakness.

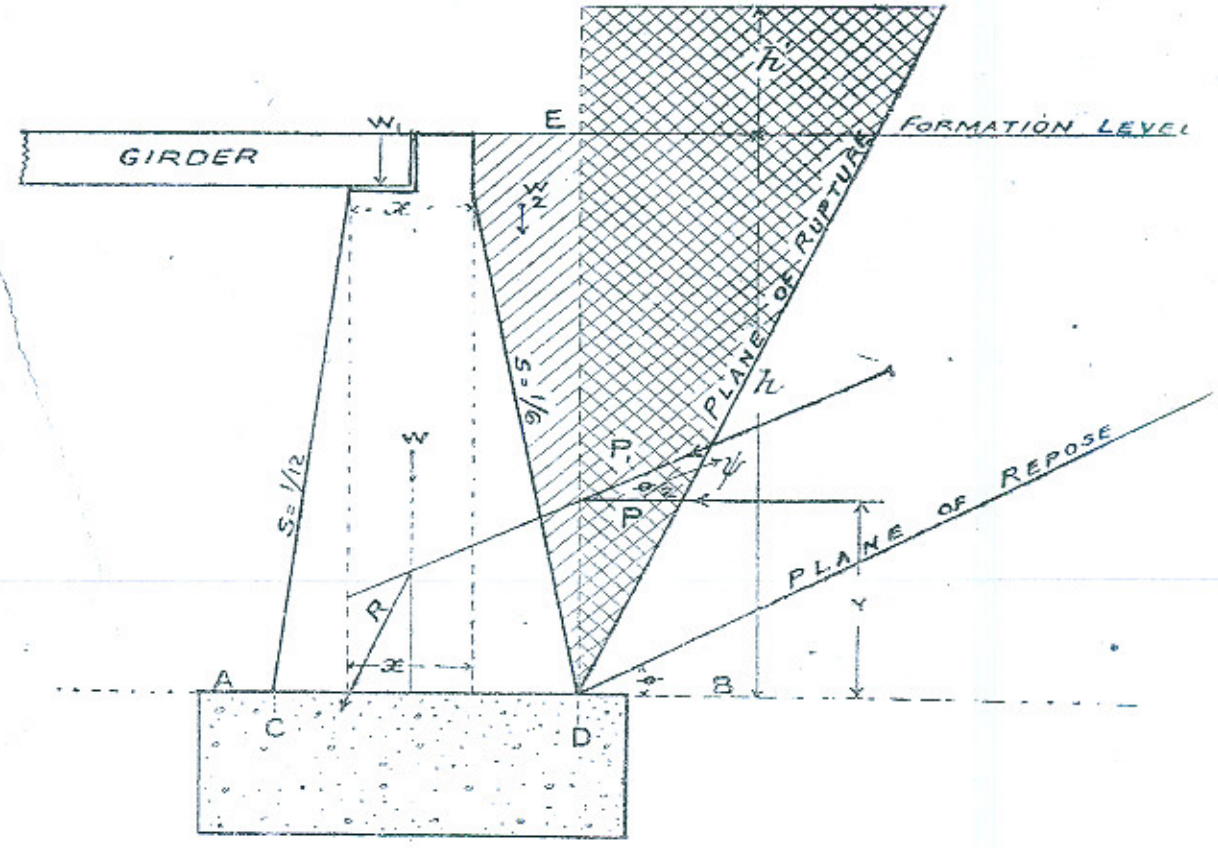
Dimensions and Weights of Standard Girders.

Clear Span.	Overall length.	Weight in tons.
6 feet. ..	8′ 6″	0·65
10 ,, ..	12′ 8″	1·24
12 ,, ..	14′ 9″	1·73
20 ,, ..	23′ 0″	3·72
30 ,, ..	33′ 0″	7·70
40 ,, ..	44′ 0″	11·60
60 ,, ..	65′ 0″	24·40

The spans up to 20 feet are made of rolled steel beams, and above that size of built up sections.

The live-load provided for is the Standard B of 1903 plus 25 per cent.

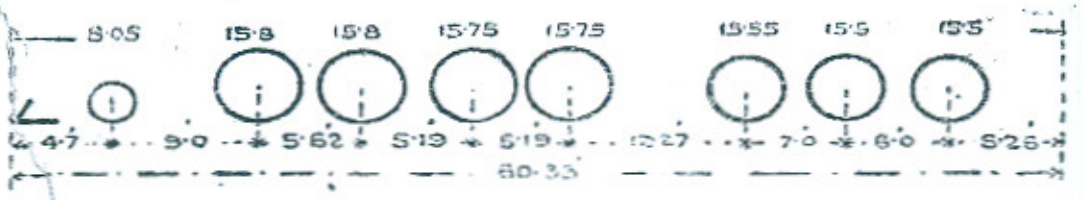
Diagram


GIRDER
W_1
E
FORMATION LEVEL
h'
$\frac{W}{2}$
x
PLANE OF RUPTURE
S = 1/12
S = 1/1
h
W
P_1
$\frac{\phi}{2}$
ψ
P
PLANE OF REPOSE
R
Y
A
C
D
B

SOME ENGINE LOAD DIAGRAMS.

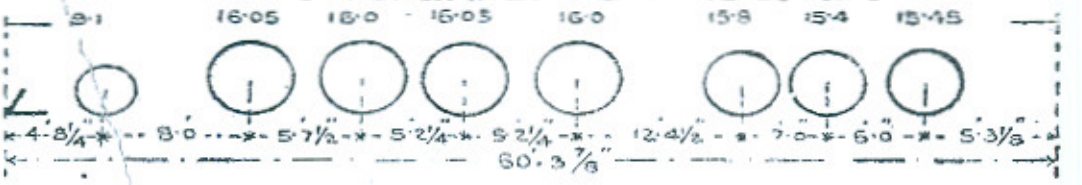
H. G. ORDINARY = 117·70 TONS

CLASS



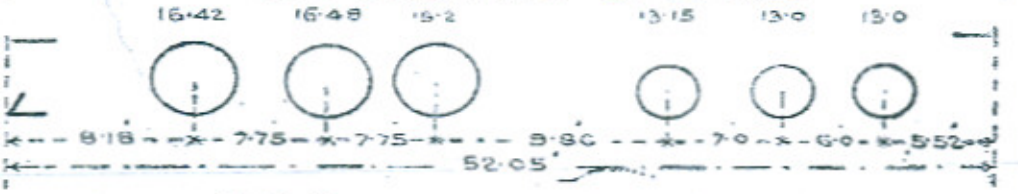
1-4-3

H. G. SUPERHEATER = 119·85 TONS



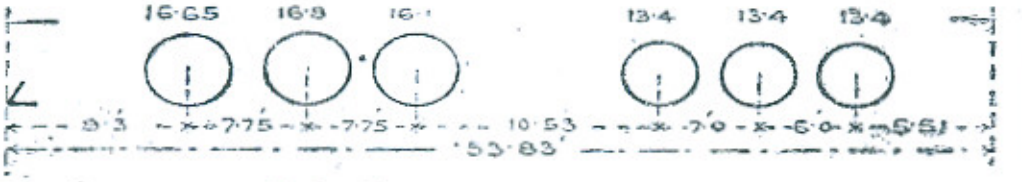
1-4-3

S. G ORDINARY = 87·25 TONS



0-3-3

S. G. SUPERHEATER = 89·75 TONS



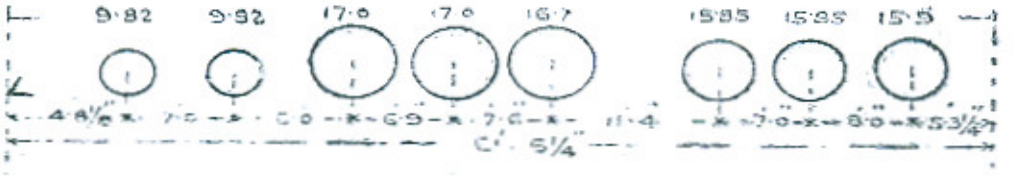
0-3-3

H.P. ORDINARY = 105·25 TONS



2-3-3

H. P. SUPERHEATER = 118·05 TONS



2-3-3

NOTE. Overall dimensions are given in black axle loads in tons and decimals and distances centre to centre in feet and decimals or inches when shown as such.

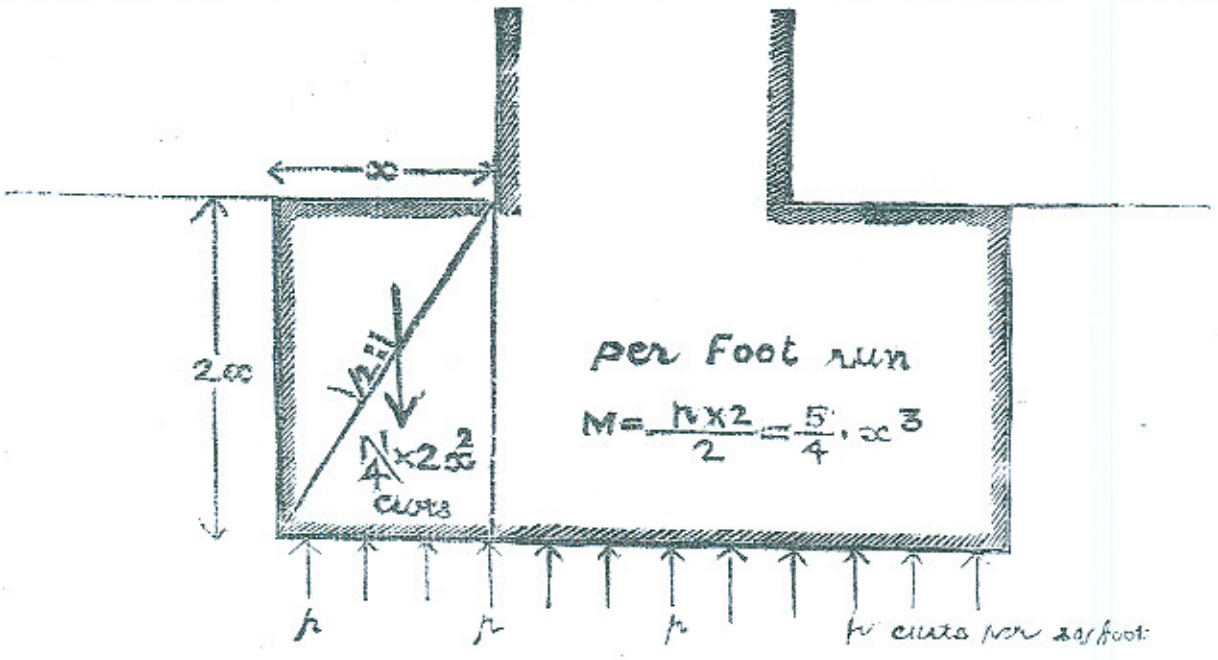
DISCUSSION.

THE AUTHOR introducing his paper said that the paper had been written chiefly for those engineers, who, like the author himself, had suddenly found themselves with bridges to construct, concerning the design of which very little information was available.

MR. F. H. BURKITT said that there was one force, small or large, which the author had left out of consideration, *viz.*, the friction at the base of the girders.

He remembered the case of an abutment 90 feet high with a girder of about 300 feet span. The friction of the girders (or resistance to rolling) in this case had a great economical effect.

He did not think engineers generally when designing the footings of abutments, went into the design as thoroughly as they should have done. The old rule of thumb, 1 : 1, was quite unsound. The footings should be calculated as cantilevers. For stresses of 25 cwts. per square foot, on the foundations and 15 cwts. per square foot extreme fibre stress in the footings, this worked out roughly as $\frac{1}{2}$: 1 slope thus.



If the footings were extended beyond this limit, they became liable to fracture.

MR. H. W. NICHOLSON related that some years ago at Rupar, 2 bridges across a torrent had been carried away and the abutments lost to sight. The bridges were afterwards rebuilt and were now standing. He had recently heard rumours that a similar case had occurred at Rupar recently and would like to know if it was true and what was the explanation.

MR. W. C. CHOPRA said he would like to know how the Author arrived at the base-width of 14 feet for a ballasted line as shown in the first paragraph on page 29. The load of the engine was concentrated on several points of two rails 5′ 6″ apart cross-wise, themselves laid on sleepers 10′ long lying at intervals apart longitudinally. It was a question if such a load could be taken as uniform on the surface of the formation, or for that matter on the length of the sleeper. Assuming uniformity of load at the base of the sleeper and taking depth of ballast to be about 2 feet, a 14 feet base would mean an angle of spread of 1 in 1. On what experience was this assumption based? Then, if the angle of spread continued below formation level the base should be more than 14 feet, probably equal to the width between wingwalls.

THE PRESIDENT said that several years ago he invented a slide rule for calculating the dimensions of retaining walls and abutment walls, and in his experience he invariably found that if a failure happened, it occurred at the toe and not near the centre? It was clear that considerable care was necessary in the design of the footing or foundations of the walls and sometimes if the ground was weak, it was advisable to extend and strengthen the footings adjoining the toe.

R. B. MAKHAN LAL said that in his opinion in designing bridge abutments the weight of the bridge girders and permanent-way should not be taken into account. It was necessary that the abutments should be built before the erection of bridge work and permanent-way thereon and it might be necessary in the future to remove the bridge girders and permanent way for repairs and for renewals or replacement by heavier girders.

MR. A. LINES said he was very doubtful whether temporary loads behind the abutments should be taken into account at all. Some engineers went as far as to say that the bridge abutments on the North-Western Railway ought to fall down. The North-Western Railway practice was to calculate in detail the sizes of the abutments for all big bridges but they did not work out in detail the abutments for little bridges except in special cases or when the foundations were not good. Furthermore, the practice was not to make any allowances for temporary loads behind abutments. For this reason, Mr. Watson's methods differed from the practice of the Railway and he would be very glad to see some of the designs for abutments prepared by Mr. Watson.

MR. D. J. MORRIS remarked that the Author's calculations were based on Rankine's Theory of Earth Pressures. This theory, it was believed, gave results, which, for any given set of conditions, were about 25 per cent. higher than the actual, as disclosed by experiment. This explained why many walls, which according to the Laws of Statics should he unstable, still stood without any signs of failure. Conditions, however,

were very variable as due to moisture or vibration which might destroy cohesion, lessen friction, add weight to the backing, and even cause it to swell, unknown forces were liable to be introduced. It was therefore unwise to cut things too close.

The discussion shewed that there was considerable difference of opinion as to where the resultant R should cut the base. Some writers recommended that R should fall within the middle third, others the middle half, and others went so far as to say that if R fell within the base all would be well. In the speaker's opinion it depended largely on the nature of the foundations, and of the backing available. If the backing was liable to change of condition as noted above a fairly large factor of safety was necessary, but logically this should be allowed for in the pressure on the back of the wall. Where, however, the foundations were soft, the point of application of R on the base should be kept as near the centre as was reasonably possible so as to distribute the pressure evenly over the base and reduce the maximum intensity at the toe. Where the foundations were good and able to stand heavy pressures without being compressed it did not matter so much, and providing the materials in the wall itself were good, greater liberties could be taken. Considering the state of our knowledge of earth pressures however, which could hardly be called exact, the speaker did not consider that R should in any case fall outside the middle half, and the main thing to be watched was the pressure at the toe of the wall.

In most designs the effect of the wings were neglected, though they undoubtedly gave support and added a further factor of safety. Such walls were however often cut down to the limit in practice, and due to unequal settlement cracks occurred. Though the wings, under such conditions might be and usually were perfectly stable, the cracks removed all support from the abutment and so if any credit were given in the calculations of the latter, the former would require very careful designing and probably considerable extra material without any economy resulting. It was therefore wise to neglect the wings in considering the abutment.

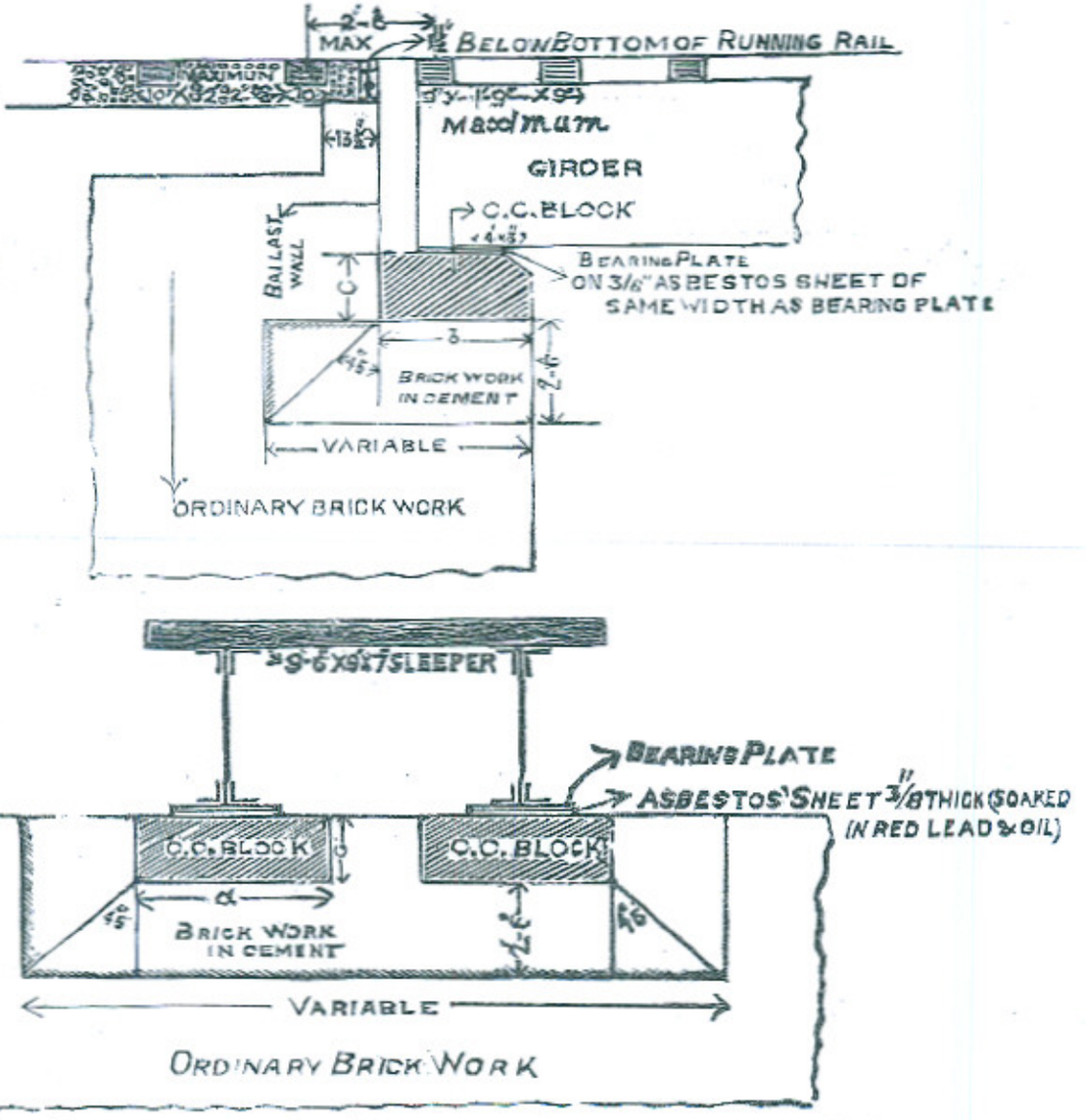
The three bridges concerning which Mr. Nicholson had heard rumours, had failed due to the floors under the bridges scouring. The work was being rushed through so that stone might be got out for the Headworks. In each case the approaches were bad and required training which could not be done for lack of time and material, the latter being all sent out to the Headworks. Heavy rain caused floods of unexpected magnitude fairly early in the monsoons and the result was that heavy scour developed at the abutments and piers which failed. In one instance at least, scour was intensified by part of the waterway being blocked by an earthen bund due to the girders not being to hand. These mishaps, therefore, had nothing to do with the strength of the abutments as calculated, and where the bridges were completed in all

respects no such trouble had occurred. The damage had since been set right at no great cost. The risks taken were justified by the fact that large quantities of stone were delivered to the Headworks where it was urgently required.

MAJOR E. P. ANDERSON, R. E., remarked that the Author had made a useful contribution to the literature on a subject about which little had been written. The Indian Railway Bridge Rules, though dealing very fully with the superstructure were silent on the subject of piers and abutments ; but some useful information on this subject could be found in Esling's paper, Inst. C. E. Vol. 165, and in a recently published book, by Williams on " Design of Masonry Structures and Foundations." The Author had made some assumptions which were at any rate open to argument. The triangle W_2 in his diagram might not act as supposed under all conditions arising in a railway bank, which may be wet or dry, extremely consolidated or the reverse. It was therefore safer to neglect it altogether or anyhow not to attach too high a value to it. The important point to consider was the train loading immediately behind the abutment. In this connection it was interesting to find, in some designs made for the Khyber Railway Construction, that for ordinary retaining walls built up to formation level the 2′ 6″ gauge engine loads were more severe on the wall than 5′ 6″ gauge because they were applied closer to the wall. The train load h′ should be applied right up to the edge of the masonry. In neglecting wind pressures the Author had apparently assumed his line as straight and level ; the wind effect on piers and abutments might be considerable even for small spans on hill lines with heavy gradients and sharp curves. Impact, and braking or traction stresses should not be mixed up. The former was usually considered as acting vertically, and represented the increase of stress above its static value produced by the running load in an infinitely short time. In the case of piers and abutments the time of application became so large, depending on the time the train took to run over one span, that impact could be neglected. The braking or traction stresses were usually considered to act parallel to the rails and to be absorbed more or less entirely by the anchorage of the track sleepers in the ballast, of which the existence of bridges on steep gradients without serious creep of girders seemed to be a proof. For economy it would often be found better to adopt an extra span and pier abutment, in preference to heavy wing walls of any sort.

MR. F. H. BURKITT said that Major Anderson had said that force W_2 in the diagram should not have been taken into account. The Punjab Public Works Department, Irrigation Branch, made practically all their wing walls battered on the earth side and took the effect of the weight of the triangle of earth resting over the back of the wall into account in their calculations. Unless this triangle was slung to a pulley and held up in position how could it be prevented from acting downwards?

The AUTHOR replying to the discussion stated that Mr. Burkitt would find that the force he referred to had not been omitted but included in the design as the following sketches would show :—

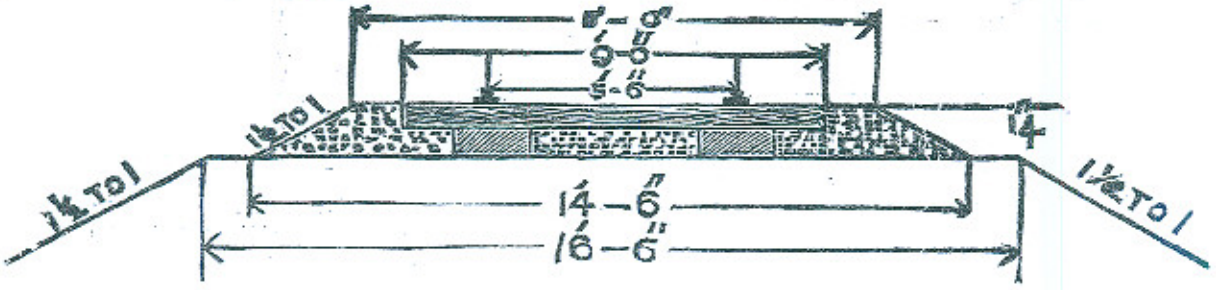


The dimensions "a", "b" and "c" varied for different spans. As regards the footings, this point had been dealt with on pages 32 and 33 of the paper.

Mr. Nicholson's enquiry having been replied to by Mr. Morris it was only necessary to add that for the sake of economy, the bridges were given light curtain walls.

In reply to Mr. Chopra, the standard minimum width of formation in embankment for the 5′ 6″ gauge track was 16′ 6″. The minimum length of the cross sleepers was 9′, breadth 10″ and depth 5 inches. It was the usual practice to use sleepers 10 feet long as this

allowed of one change should the bearing surface under the rails be damaged. The depth of the ballast under the sleepers was limited to 9 inches and the width was usually extended 1 foot beyond the sleepers at each end so as to box them in, and between sleepers the top of the ballast was level with the top of the sleepers except where it was desirable to provide for drainage down the centre of the track.



The above was a typical dimensioned sketch, the actual bearing points being immediately under the rails where the packing for a width of 18" to 2 feet under each rail had to be carefully and properly done; represented on sketch by hachures. The sleeper had to be taken as a beam carrying the load which was transmitted on to the ballast; this latter became more compact and homogeneous and hence the base width of 14' 6" or roughly 14 feet was taken as the effective width over which the load was evenly distributed, and this width was taken vertically downwards, which assumptions might not be absolutely correct but appeared to serve the purpose in the absence of more definite knowledge of the actual incidence and distribution.

Pages 32 and 33 of the paper in which the question of the footings has been dealt with under the heading of foundations, covered the points raised by the President. The author disagreed with the opinion of R. B. Makhan Lal that the weight of the bridge girders and permanent-way should not be taken into account. Girders and permanent-way material were standardized to suit the loads and spans and allowed for a margin or factor of safety. If these items were omitted it would not be possible to efficiently design the abutments and piers, as it was necessary to know as accurately as possible the resulting pressures on each square foot of area of the foundations in order to keep the pressures within the limits permissible for the class of soil on which the structures were to be built. When it became necessary in the future to change the girders to provide for heavier loads, the correct procedure was to work out the problem again and see whether the existing dimensions were suitable or required strengthening.

In reply to Mr. Lines, if sufficient time were available to allow the earth to settle and consolidate behind the abutments before the line was brought into use, the question of the temporary loads behind the abutments would not be so important. In the cases dealt with on the Doraha-Nalagarh Railway however, the working had to be started immediately the bridge-work was ready and while some of it was still

green. As pointed out by Mr. Morris, conditions were very variable, and it was not therefore advisable to cut things too fine.

Mr. Burkitt had already disposed of Major Anderson's objection to the force W_2 in the diagram being taken into account, and it was therefore unnecessary to make any further remarks on it. His remarks about impact and the effect of the application of brakes being mixed up were not clear. It had nowhere been stated in the paper that these forces were the same. The paper dealt with straight and level bridges. When the bridge was on the curve (which was not done unless unavoidable or likely to increase cost unduly), provision must be made for the forces set up due to curvature and super-elevation of the outer rail, and in cases of gradients of a steep nature the effect of the grade would have to be considered.

He agreed that under certain conditions it was more economical to extend the span than to put in heavy end abutments, but as had already been remarked, the pier abutments must be correctly designed.