PAPER No. 149.

METHODS OF CALCULATION AND ANALYSIS OF STRUCTURES EMPLOYED ON THE N. W. RAILWAY.

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In connection with the running of the new heavy engines on the North Western Railway it became necessary to analyse the strength of abutments and piers of girder bridges and also of arch bridges to see whether they would be strong enough to take the heavier axle loads. It is very probable that the bulk of our bridges on the older sections of this Railway were designed for 12 ton or even smaller axle loads. Technical paper No. 122 on the revised Bridge Rules of 1903, by Mr. C. W. Hodson, M. Inst. C. E., the then Director of Railway Construction on page 3, gives the following statement of maximum axle loads current in the past :—

Gauge.	1875.		1882.	1886	1891.	1892.
	Light.	Heavy.				
5′—6″ Maximum axle loads engines	$7\frac{1}{2}$	12	12	14	15	15 tons.
Maximum axle loads wagons..	$7\frac{1}{2}$	8	8	9	11	12 tons.

Statement 'A' gives the dates of openings of the various older sections of the North Western Railway.

Statement 'B' gives the dates of introduction of various classes of engines on the North Western Railway.

STATEMENT 'A'.

Portion of North Western Railway opened before 1875;

		Date of opening.	Miles.
	Double Lines.		
1.	Karachi City to Karachi Cantt. ..	13-5-61	2·30
	Single Lines.		
2.	Kotri to Karachi City ..	13-5-61	105·03
3.	Amritsar to Lahore ..	10-4-62	34·09
4.	Delhi to Ghaziabad (*a*) ..	1-8-64	13·96
5.	Lahore to Khanewal ..	24-4-65	175·17
6.	Khanewal to Multan Cantt. ..	24-4-65	30·5
7.	Ghaziabad to Meerut City ..	18-4-67	29·41
8.	Beas to Amritsar ..	1-11-67	26·23
9.	Meerut City to Ambala Cantt. ..	1-1-69	123·72
10.	Ambala Cantt. to Ludhiana ..	12-10-69	71·46
11.	Jullundur Cantt. to Beas ..	15-11-69	25·53
12.	Phillaur to Jullundur Cantt. ..	1-4-70	26·12
13.	Multan Cantt. to Muzaffarabad ..	21-8-70	6·89
14.	Ludhiana to Phillaur ..	14-10-70	8·28

STATEMENT 'B'.

Dates of introduction of various classes of engines run on North Western Railway.

Class of engine.	Date when 1st run.	Maximum axle load in tons.
L. L.	October 1885	10·1
H. L.	December 1885	12·3
T. G.	1890	14
H. B.	September 1892	13·5
P.	September 1896	14
K. R.	August 1897	14·4
T. A. A.	December 1901	18·2
M.	October 1902	15·3
S. T.	November 1902	14·45
M. Modified	December 1902	17·2
S. P.	February 1905	15·75
S. G.	June 1905	15·2
H. P.	April 1906	16·05
H. G.	January 1907	15·8
S. G./C.	July 1908	17·7
A. P.	August 1908	17·5
P. T.	June 1909	15·5
G. P.	October 1910	17·5
S. P./S.	May 1912	16·9
H. G./S.	February 1915	16·05
H. P./S.	June 1915	17·0
S. G./S.	August 1920	16·8
N.	March 1921	19·0
H. S. T.	July 1924	17·0
M. A./S.	January 1924	17·5
G. A./S.	December 1925	19·5
X. C.	October 1928	19·7

This statement has been extracted from a statement showing the general conditions of locomotives and boilers for the half year ending 31st March 1929.

It will be noticed from these two statements that the axle loads of our engines have been progressively increasing and that 16 to 17 tons axle loads have been running over masonry bridges designed presumably for very much lighter axle loads of 10 to 12 tons. To keep pace with this progressive increase in axle loads our Bridge Department has no doubt been renewing weaker girders with girders of proper strength according to definite programmes, but requisite systematic attention does not seem to have been paid in the past to the extra stresses that must have been induced in the masonry and foundations of the bridges whenever increased axle loads were run over them. Statement 'C' shows the calculated stresses in some of the various types of masonry bridges on the North Western Railway under the existing loads and the stresses that are likely to come over them under the new $22\frac{1}{2}$ tons and 28 tons axle loads.

STATEMENT 'C'.

Existing loads.

Divn.	Bridge.		Mileage.	Max: Comp: at bed Tons/Sft.	Max: Comp: at founds. Tons/Sft.	Max: Comp: in archring Tons/Sft.
	Kind.	Clear Span.				
KYC ..	Girder	40′-0″	308/13-14	4·80	3·70	..
Do. ..	Do.	19′-8″	377/2-3	7·40	3·92	..
LHR ..	Do.	15′-0″	729/23	4·00	1·23	..
DLI ..	Do.	25′-6″	157/1-2	4·13	2·07	..
Do. ..	Do.	10′-0″	78/2	2·27	2·13	..
Do. ..	Arch	10′-0″	212/-	3·20	1·82	9·34
Do. ..	Do.	6′-0″	150/12-13	3·15	1·80	6·02

M. L. Loads.

Divn.	Bridge.		Mileage.	Max: Comp: at bed Tons/Sft.	Max: Comp: at founds. Tons/Sft.	Max: Comp: in archring Tons/Sft.
	Kind.	Clear Span.				
KYC ..	Girder	40′-0″	308/13-14	7·40	4·60	..
Do. ..	do.	19′-8″	377/2-3	11·87	5·70	..
LHR ..	do.	15′-0″	729/23	5·25	1·56	..
DLI ..	do.	25′-6″	157/1-2	5·48	2·61	..
Do. ..	do.	10′-0″	78/2	2·95	2·71	..
Do. ..	Arch	10′-0″	212/-	3·84	2·16	11·9
Do. ..	do.	6′-0″	150/12-13	3·57	2·01	6·90

H. M. Loads.

Divn.		Bridge. Kind.	Bridge. Clear Span.	Mileage.	Max: Comp; at bed Tons/Sft.	Max: Comp; at founds. Tons/Sft.	Max; Comp: in archring Tons/Sft.
KYC	..	Girder	40′-0″	308/13-14	9·40	5·65	..
Do.	..	do.	19′-8″	377/2-3	13·57	6·72	..
LHR	..	do.	15′-0″	729/23	6·80	1·95	..
DLI	..	do.	25′-6″	157/1-2	7·27	3·28	..
Do.	..	do.	10′-0″	78/2	3·74	3·24	..
Do.	..	Arch	10′-0″	212/-	not	worked	out
Do.	..	do.	6′-0″	150/12-13			

Arches, abutments and piers had been known to fail in the past and it was therefore reasonable to anticipate further failures with the proposed introduction of the 22½ tons (M. L. Main Line Standard) and 28 tons (H. M. Heavy Mineral Standard) axle loads producing considerable overstresses as they would be. It became therefore imperative to set our house in order and to examine this question in detail before allowing 22½ ton engines to run over them. Consequently it was decided to analyse all girder and arched bridges of 6 feet span and over. Unfortunately most of our older bridges had no completion plans. Therefore information had to be called for from the Divisions as regards the condition of masonry and the nature of founds. Divisions were also asked to supply drawings or sketches showing sections of arch rings and piers. It was found difficult to obtain sections of abutments without dislocating traffic, but successful experiments were made by jumping holes in masonry either by hand or by a pneumatic drilling machine, to get over this difficulty. This method though costly was found very useful where traffic was not to be disturbed and is now employed whenever necessity arises in important cases on this railway.

Methods of Calculations.

For calculations dealing with the strength of a railway bridge and for the determination of the loads for which such a bridge may be used, reference is invited to the Government of India Rules for the opening of a railway, Revised Chapter VII of 1926. These are briefly summarized as below :—

For the purpose of computing the maximum stresses in any girder or member of a bridge each of the following items have to be taken into account :—

(*a*) Dead load.

(*b*) Live load.

(c) Impact.

(*d*) Forces due to curvature or eccentricity of track.

(*e*) Deformation stresses.

(*f*) Secondary stresses.

(*g*) Wind pressure and other forces causing lateral deflection.

(*h*) Longitudinal forces.

(*i*) Temperature effects.

(*j*) Erection stresses.

Dead Load carried by a girder or member consists of that portion of the weight of the superstructure and the fixed loads carried thereon, which is supported wholly or in part by the girder or member including its own weight.

For the **Live Load** three standards have been laid for the 5′-6″ Gauge :—

1. Standard H. M. for 28 tons axle loads and a train of 2·3 tons per foot behind the engine.
2. Standard M. L. for $22\frac{1}{2}$ tons axle loads and a train of 2·3 tons per foot behind the engine.
3. Standard B. L. for 17 tons axle loads and a train of 1·5 tons per foot run behind the engine.

Longitudinal Forces.—Provision has to be made for the stresses due to the tractive effort of the live load and the braking effect resulting from the application of brakes to such load while passing thereover, these forces being considered as acting at rail level.

The amount of the tractive effort on one track is ascertained by multiplying $1\frac{3}{4}$ times the maximum end shear due to the live load on that track by a factor equal to $\frac{20}{L + 75}$ where L= span in feet. The factor is not to exceed 0·15 as a maximum.

The braking effect is similarly determined by using a factor equal to $\frac{12}{L + 90} + 0{\cdot}075$, also limited to a maximum of 0·15.

No increase for impact effect is made on the stresses due to longiudinal forces.

Special rules for arch bridges are summarized as follows :—

The elastic theory usually adopted in the case of steel and ferro-concrete arches is not strictly applicable in the case of the ordinary masonry arch built under a railway. For calculating stresses in arched masonry bridges the ordinary method described in standard text books may be followed. The ordinary method of locating the lines of pressure is based upon assumptions which cannot be verified but which have been proved by long experience to be sufficiently accurate for all practical purposes.

Impact.—Increment for Impact to be allowed on the whole of the moving load 'M' may be calculated for spans up to and above 50 feet respectively from the formulae.

$$I = \tfrac{1}{2} \times \frac{120}{90 + \left(\frac{n+1}{2}\right) L} M, \text{ and } \tfrac{1}{2} \times \frac{300}{300 + L} M$$

provided there is cushion of earth, ballast, etc. more than 2 feet thick below the bottom of sleepers.

If however the cushion is not more than 2 feet, increments for impact should be calculated from the formulae.

$$I = \frac{120}{90 + \left(\frac{n+1}{2}\right) L} . M_T \quad \text{and,} \quad \frac{300}{L + 300} . M_T$$

Maximum permissible stresses must not be greater than 1/5th of the ultimate strength of the material used.

The co-efficient of friction of a mortar joint is to be taken as decimal seven (·7). The resistance in shear of a lime mortar joint may be taken at not more than 300 lbs. per square foot.

These rules were exactly the same as laid down in 1903 and were not revised in 1926. It will be noticed from a perusal of these that the main factors, affecting the designs of arched bridges, and abutments and wing walls of Girder Bridges, were still left to each individual Engineer to decide for himself. For instance the pressure of the back fill, the amount of surcharge on the same due to train load; determination of safe foundation pressure, depth of foundations, earth cushion, safe masonry stresses (in compression, tension, shear or sliding), the best shape of abutments and piers whether with or without batter, and the effect of tractive effort and braking forces on them, were some of the factors which this Railway had to determine for themselves when designing new bridges or analysing the strength of existing bridges with a view to determining whether they were safe or otherwise under the new heavy axle load engines. The process has been a sort of evolutionary one for the last four years and the object of this paper is to present to the members of this Congress the methods of calculations and the assumption of data now in use on this Railway and how they have been arrived at. The easiest way would be to give actually worked out examples of these.

Arched Masonry Bridges.—For simplicity of form for small spans they are built of a segmental shape *i.e.* rise $=\frac{1}{8}$ span.

Design and calculation of a 12 ft. arch for H. M. loading are given *vide* Plates No. III, IV, V, VI and VII. For simplicity of calculations certain assumptions are necessary to investigate the worst conditions of loading.

Live load is taken from actual wheel loads and is = 8 cwts. per sq. ft. Dead load is that of 3 ft. cushion and P. Way (*vide* Plate No. III).

The spread of the load along the barrel of the arch is taken at 45° from the edge of sleepers (*vide* Plate No. III). Weight of earth is taken 1 cwt./cft. Weight of masonry is taken 1 cwt./cft. Line of thrust is drawn graphically by Fuller's method given in G. F. Charnock's Graphical Statics, both when the arch is fully and symmetrically loaded with the live load on (*vide* Plate No. III) and also when it is eccentrically loaded with the live load on half the span (*vide* Plate No. VII), and in both cases is kept within the middle third of the arch ring. For the purpose of drawing the stress diagram the live load including impact is plotted into an equivalent height of dead load.

Safe Stress—In cement masonry this is taken as 10 tons/sq. ft. and in line masonry as 5 tons/sq. ft.

Minimum cushion below bottom of sleepers is taken as 3 ft. and the increment in live load due to impact in such cases is taken as 20 per cent of the live load.

Where cushion is less than 3 ft. as in the case of some of the existing bridges the impact is taken as $I = \dfrac{120}{90 + \left(\dfrac{n+1}{2}\right) L}$

No impact however is taken into account when designing the abutments and foundations.

Angle of repose of the back fill is taken as 30°. The longitudinal thrust of this fill is calculated by Rankine's formula.

$$\tfrac{1}{2}\, wh^2 \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$$

Girder bridges.

Design and calculations for a 40 feet girder abutment under H. M. loading is given (*vide* Plates No. VIII and IX and Appendix III).

Live load is 109·87 tons=the maximum end shear, taken from the tables.

Dead load includes weight of girders.

P. Way etc. = 18·61 tons for the span. Impact is not allowed for as it is assumed that it must be dissipated through the sleepers and girders. Surcharge is taken as 5 feet above rail level and the angle of repose is taken as before, 30°. (Calculations given *vide* Appendix I).

The horizontal thrust of the fill is calculated by Rankine's formula and an allowance for cohesion is made by deducting 30 per cent. according to Rebhann's. This in practice amounts to assuming an angle of repose equal to 37° in the case of new bridges. In the case of older abutments, however, where the earth gets time to get consolidated, it is reasonable to assume a bigger angle of repose and therefore 45° is usually taken as the angle of repose in these cases. This thrust is assumed as uniformly distributed along the whole length of the abutment.

Data assumed and difficulties.

In the data assumed above it is recognized there are certain elements of uncertainty, *e.g.* :—

(*a*) In the value of the angle of repose of the back fill and the amount and position of the resultant thrust.

(*b*) In the value of Tractive effort and Braking force and its point of application, whether on the rails or on girder bed.

(*c*) The surcharge due to the live load and its amount.

(*d*) The angle of spread of the loads both in the fill and in the masonry.

(*e*) Permissible safe working stress in brick work.

(*f*) Permissible safe foundation pressures.

(*g*) Impact.

We will now discuss these one by one.

(*a*) **Angle of repose.**

The value of the angle of repose depends upon a number of factors, as for example the kind of soil, its composition, its degree of compactness, the amount of moisture in it and the effect on it of various climatic conditions. These factors vary so widely that the value of ϕ according to different authorities varies from 18° for soil saturated with water to 45° for compact clays. Moreover the same material may change from day to day in character owing to changes in moisture and from point to point owing to cohesion being more lower down than at the top.

For the calculation of the amount and position of the resultant earth pressure on an abutment, many theories have been propounded depending on the assumed behaviour of the soil on the one hand like a granular mass with no internal friction between its particles, and on the other, more or less like an organized mass with a range of value of adhesion and of the angle of internal friction depending upon the conditions enumerated above.

As already noted these conditions vary so widely that it is difficult to determine with certainty the exact effect that each of these has got on the amount and position of the resultant thrust. Consequently owing to the variable nature of the factors involved the estimates of earth pressures can only be rough approximities. Rankine's theory is simple and easy of application and for that reason has been adopted by us.

(b) Tractive Effort and Braking effects.

There is not the slightest doubt that theoretically these forces must develop at rail level, but the amount and what proportion of the same is transmitted to the piers and abutments at bed stones is uncertain.

Were the full amount as laid down by the Bridge Rules to be taken into account, the designs would result in unnecessarily thick piers and abutments if conditions of stability have to be satisfied. Experiments have therefore been undertaken on this Railway to determine these forces if possible, and the results which would be interesting, are anxiously awaited. So far the experiments performed though not exhaustive, show that part only of the forces developed at rail level is transmitted to the abutments or piers at girder bed level but its amount depends on the rigidity of rail connections and type of flooring used. Whether the rigidity of Permanent-way connection with girders and the type and condition of the bearings of girders has any effect has not so far been investigated. Pending final results of these experiments the maximum amount of tractive effort and braking forces transmitted to the pier and abutment on the top of bed stones is assumed to be 67 per cent. of the amount given in the Bridge Rules in the case of open floored bridges and 50 per cent. in the case of ballasted floors.

(c) Surcharge.

The question of surcharge due to axle loads of a train and the effect of each on the abutment is not quite definite. The following among others are some of the uncertain factors involved :—

(*i*) The law of distribution of different axle loads through rails to the sleepers.

(*ii*) The angle of spread of the load through each sleeper into the embankment.

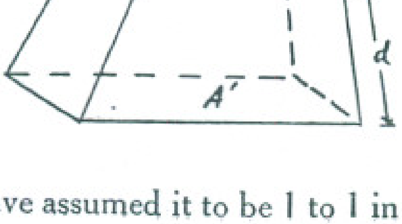
(*iii*) The effective length of the abutment that comes into action in resisting surcharge effects.

(*iv*) The effective distance behind the abutment.

(*v*) The effect of the return or wing walls.

(d) Angle of spread of loads through earth and masonry.

If a load were applied uniformly over an area A at the horizontal surface of any ground, then at any plane A′, parallel to the surface and at depth d below it the load will be distributed over an area A′ larger than A. This increase in area depends on the varying condition of the soil already enumerated in (a) above. The angle of spread of loads through earth fills has been taken by various authorities as from ½ horizontal to 1 vertical to 1 horizontal to 1 vertical and we have assumed it to be 1 to 1 in our calculations.



The angle of spread of the load through masonry is also largely indeterminate and there does not appear to be much experimental evidence available in this country, which would enable us to determine this with any degree of accuracy. The problem deserves greater attention than hitherto paid to it, as it figures prominently in the general consideration of the area of masonry structures effected under a system of concentrated loads, and upon which depends largely the design of masonry abutments and piers.

(e) Permissible safe stresses in brickwork.

The permissible safe stress in brickwork varies with the quality of bricks, the quality of mortar used and the workmanship, and all these vary in various localities. And as the variations are so wide it appears difficult to lay down any set values that will meet all conditions and localities. This will be apparent from the fact that seven samples of brick-work-in-lime, from various localities on this Railway were sent to the Alipore Test House for testing purposes, and the results gave crushing strengths varying from 29 tons per sq. ft. to 77 tons per sq. ft., a very wide variation indeed. Similarly two samples of brickwork in cement gave the crushing strengths of 137 and 173 tons per sq. ft. respectively.

Consequently, it appears necessary that brickwork tests must be performed before assumptions are made for the safe working stresses. The assumed safe stress on this Railway for brickwork in lime mortar as 5 tons per square foot and for brickwork in cement as 10 tons sq. ft. appears fully justified as a result of such tests.

Analysis of older bridges and arches shows that stresses considerably more than assumed above as safe must be developing, but this fact cannot form an argument that higher safe stresses should be allowed in our new designs also. All that can be inferred from such analysis is that the older structures are standing because the crushing limit of masonry has not been reached so far. The factor of safety no doubt has been reduced but the reduction is such that could be safely accepted in the case of masonry structures which have withstood the test of time, for the simple reason that such structures give ample indication and warnings before they collapse or become dangerous. How far this reduction in the factor of safety can be accepted would appear a matter of judgment. Analysis of numerous bridges on this Railway so far has led us to conclude that 10 tons per square foot for masonry in lime in the case of older bridges is about the limit that could be safely worked up to.

(f) Permissible safe foundation pressures.

Permissible safe foundation pressures depend on several factors such as :—

(*i*) Nature and condition of soil.

(*ii*) The depth of foundation.

(*iii*) The area of the base of structure.

(*iv*) The maximum differential settlement the structure can stand.

The information at present available concerning the character and behaviour of different soils is inadequate and the interpretation of the observed facts is arbitrary. In fact foundation problems throughout are of such a nature that a strictly theoretical mathematical treatment will always be impossible. Definite values of the bearing capacity of various soils cannot be stated with accuracy because of the variations in character and condition of the same kind of soils and consequent difficulty in classifying them. Moreover the ability of the soils to sustain loads depends not only upon its character but also upon the amount of water it contains and the degree to which it is confined in position. Hence it is that direct tests of the capacity of the soil to support the loads coming upon a foundation are very desirable.

In this connection a paper written by Charles Tarzaghi on the "Science of Foundations—its Present and Future" and the subsequent discussion printed in the proceedings of the American Society of Civil Engineers would be found very interesting and instructive. On this Railway several soil-bearing pressure tests have been carried out and the method described is given in Appendix II.

As a result of several of these it has been laid down that for the portion of the country through which the bulk of this Railway passes foundation pressures in average soil, ***i.e.***, a reasonable proportion of sand and clay must not exceed 1·0 ton per square foot in the case of buildings where *differential* settlements are liable to be comparatively greater and more harmful, and 1·5 tons per square foot in cases of open found-bridges where differential settlements are not great and less harmful.

For the more important structures, actual soil-bearing pressure tests at site must be carried on, on the spot before designs are undertaken.

As in the case of pressures in masonry of older bridges, analysis shows that pressures on the foundations of older bridges also come out to be very much higher than those laid down above. The reason probably is that soils under the foundation have consolidated during the course of time and that the structures are standing with a reduced factor of safety. It is again a matter of judgment as to how much reduction in the factor of safety is possible. On this Railway as a result of analysis of several existing bridges it has been found that foundation pressures in the case of existing bridges, up to 2·5 tons per square foot need not cause any alarm.

(g) Impact.

There is little experimental evidence to determine the effect of Impact of moving loads on the abutments, piers and foundations of girder and arch bridges. It is probable that most of the Impact is dissipated through the cushioning effect of the sleepers and girders before it reaches the masonry, but some residual effect must be passing through these and reach the few top layers of masonry and induce extra stresses tending to shake up the masonry. These extra stresses cannot be calculated but are met with by building in cement the top 2 feet of masonry of piers and abutments immediately below the girder bed stones. It is however very doubtful that any effect of the Impact does ever reach the floor level or on foundations. Consequently in calculations for these no extra allowance in stresses due to Impact is allowed for. Similarly in the case of arches, the cushion on the top of thecrown does absorb bulk of the impact and it appears therefore reasonable to assume that with a cushion of 3 feet between the top of the crown and the bottom of sleeper an increase of 20 per cent. in stresses in the arch ring on account of Impact is ample, no increase being allowed for in abutment piers and foundations. Definit experimental evidence in this case, however, is not forthcoming.

APPENDIX I.

Calculations for surcharge to be allowed for H. M. Loading.

Data.

1. Axle Load =28 tons.
2. Distance between axles=5·6 ft.

The standard length of an abutment for single track =20 ft.

The surcharge is assumed to be distributed uniformly over the whole of this 20 feet length for purpose of these calculations. This assumption of course is true only for a height of bank of 5′-6″ or more on the supposition that the angle of spread through the fill is 1 to 1.

The weight of the earth is taken as 112 lbs. per cubic foot.

Live Load per foot length of track=28/5′-6″=5 tons.

Distributing it over 20 feet length of abutment we have, load per foot$=5\times\frac{2240}{20}=560$ lbs. =5 cwts.

Converting this into earth units we have the corresponding height of surcharge above rail level, $5\times\frac{112}{112}=5$ feet.

APPENDIX II.

Procedure to be adopted in carrying out pressure settlement tests.

(*a*). A steel plate 12″×12″×1″ should be fixed to the bottom of a pile 12″×12″ or slightly less in section.

(*b*). Depending on the nature of the strata and of the proposed structure, the level below ground line to be adopted for the test should be that at which the footings of the proposed structure will probably be founded.

(*c*). The smallest practicable pit (about 3 feet square) with sides as near vertical as the soil will stand, should be dug down to within 2 feet of the level at which the test is to be carried out. In the centre of this pit a hole should be made with vertical sides 2 feet deep and not larger than 18″×18″ to receive the foot of the pile. If the soil will not permit of this being done a box of sheet iron or wood 18″×18″×24″ should be inserted and the space round it back filled and tamped to the hardness of natural soil.

(*d*). The loading platform on top of the pile should be as shewn on Plate I.

It will be seen from the above figure that the loading block is steadied by the four lever beams on which the loading platform is supported and whose outer ends rest on fixed blocking. Under this arrangement is developed a steady downward pressure on the plunger.

(*e*). The pile should be loaded gradually by half ton increments up to a maximum of 5 tons.

(*f*). After each half ton increment the pile should be allowed to rest until the settlement ceases entirely for all practical purposes. This will usually occur within 2 or 3 days.

(*g*). Settlement readings should be taken after each increment of load and after that at intervals of 12 to 24 hours as found necessary until settlement ceases.

(*h*). A load-settlement graph with notes as regards time intervals between successive load increment should be prepared as in Plate II.

(*i*). A trial pit or tube should be sunk near the test pit to a depth below the bottom of the test pit equal to 1·5 times the width of the widest footing in the proposed structure and the strata should be recorded on a trial pit section. This is to see if there are any highly unstable strata within the zone of the pressure bulb.

(*j*). The number of tests and trial pits necessary for a proposed structure will depend on the size and nature of the structure and the variations in the soil.

APPENDIX III.

Calculations for a 40 feet Girder abutment under H.M. Loading.

Live and dead loads on the span.

Live Load without impact taken from shear table	=	219·74 tons
Weight of girders including bearings	=	14·21 „
Weight of Permanent-way (taken at 0·1 ton per foot run)	=	4·40 „
Total	=	238·35 „
Load on one abutment $=\frac{238 \cdot 35}{2}$	=	119·175 „
Say	=	119·2 „

acting at the centre of bearing.

Length of abutment at bed level=20 feet.

Sectional elevation of the abutment is given in Plate IX, Fig. 2.

Taking 1 to 1 distribution in cement masonry and $\frac{1}{2}$ to 1 in lime masonry the maximum length of abutment over which the load can be distributed is $= 20'\text{-}10\frac{11''}{16}$ but actually the length of the abutment is only 20 feet. Therefore load on one foot length of abutment$=\frac{119 \cdot 2\times 20}{20}$

$=119 \cdot 2$ cwts.

Maximum Braking or Tractive effort effects on one foot length of the abutment as per Bridge Rules, at Rail Level$=\frac{219 \cdot 74}{2}\times\frac{7}{4}\times\frac{1}{2}\times \cdot 15\times\frac{20}{20}=\frac{230 \cdot 73}{16}=14 \cdot 42$ cwts, assuming that it is uniformly distributed on the whole length of each of the abutments. But for our purpose, 2/3 of this amount is assumed to be functioning at the top of the bedstone, and, is equal to $14 \cdot 42\times 2/3=\frac{28 \cdot 84}{3}=9 \cdot 61$ cwts.

Earth pressure —

Axle load	=28 tons.
Distance the axles are apart	=5·6 feet.
Length of abutment	=20 feet.

For simplicity in calculations it is assumed that weight of one cubic foot of masonry =weight of one cubic foot of earth =1 cwt.

Then surcharge $=\frac{28}{5 \cdot 6}\times\frac{20}{20}=5$ feet.

Angle of repose of soil is assumed as 30.° Earth presure (by Rankine's formula) Fig. 3 Plate IX $= \frac{1}{2}W \times H \times (H + 2h) \frac{(1 - \sin\phi)}{(1 + \sin\phi)} =$ $\frac{1}{2} \times 1 \times 15\,(15 + 2 \times 6) \left(\frac{1-\frac{1}{2}}{1+\frac{1}{2}}\right) = \frac{1}{2} \times 15 \times 27 \times 1/3 = 135/2$ $= 67{\cdot}5$ cwts.

Deduct 30 per cent. of Earth Pressure to make allowance for cohesion as per Rebhann's theory $= 67{\cdot}5 \times {\cdot}7 = 47{\cdot}25$ cwts.

Say $= 47{\cdot}3$ cwts.

Point of application of resultant Earth Pressure above point 0 *i.e.* above bed level $= H/3 \times \frac{(H + 3h)}{(H + 2h)} = \frac{15 \times 33}{3 \times 27} = \frac{55}{9} = 6{\cdot}11$ feet.

The width of the abutment at bed stone level is governed by

(1) the bearing of the girders ;

(2) the width of the ballast wall.

The former is $= \frac{44-40}{2} = 2$ ft.

The latter has been worked out to be $= 3{\cdot}38$ feet.

Clearance $= 4$ inch.

Hence total width at bed stone level $= 5{\cdot}71$ feet. In analysing the stability of the abutment at bed level, consider forces on one foot length of abutment :—

Resultant passes through the middle third (assumed) (bearing and ballast wall being already fixed).

To determine the most economical batter to satisfy the condition that the resultant just passes at the extreme edge of the middle-third :—

Take moments about the point 0 (see Fig. 1, Plate VIII).

Assume CD = X ft.

	V. Force.		Arm.	Moment.
Live and dead loads ..	119·2	×	4·35 =	518·5
Abutment portion $\frac{8{\cdot}31 \times X}{2}$				
	$= 4{\cdot}16\,X$	×	$\left(5{\cdot}71 + \frac{X}{3}\right)$ =	$1{\cdot}39\,X^2 + 23{\cdot}7\,X$
Abutment below girder seating $(2{\cdot}33 \times 9{\cdot}81) =$	22·9	×	4·54 =	104·0
Ballast wall, surcharge and masonry below $3{\cdot}38 \times 21{\cdot}0 =$	71·0	×	1·19 =	84·5
Tractive or Braking Effort effects	(9·6)	×	9·81 =	94·2
Earth Pressure =	(47·3)	×	6·11 =	289·0
Total	$= 4{\cdot}16X + 213{\cdot}1.$			$1{\cdot}39X^2 + 23{\cdot}7\,X + 1090{\cdot}2$

Distance of Resultant from $O = \frac{1{\cdot}39X^2 + 23{\cdot}7\,X + 1090{\cdot}2}{4{\cdot}16\,X + 213{\cdot}1} = \frac{(5{\cdot}71 + X)2}{3}$

i. e. $4{\cdot}16X^2 + 71{\cdot}1X + 3270{\cdot}6 = 8{\cdot}31X^2 + 426{\cdot}2X + 2433{\cdot}6 + 47{\cdot}45X$

or $4{\cdot}15X^2 + 402{\cdot}55X - 837 = 0$, $(ax^2 + bx + c = 0)$

$$\therefore x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

Solving the Quadratic.

$$\therefore X = \frac{-402{\cdot}55 \pm \sqrt{(-402{\cdot}55)^2 + 4 \times 4{\cdot}15 \times 837}}{2 \times 4{\cdot}15}$$

$$= \frac{-402{\cdot}55 \pm \sqrt{162046{\cdot}50 + 13894{\cdot}2}}{8{\cdot}3} = \frac{-402{\cdot}55 + \sqrt{175940{\cdot}70}}{8{\cdot}3}$$

$$= \frac{-402{\cdot}55 + 418{\cdot}97}{8{\cdot}3} = \frac{16{\cdot}42}{8{\cdot}3} \text{ (neglecting the minus)}$$

$X = 1{\cdot}98$

$\therefore$ Batter $= \frac{8{\cdot}31}{1{\cdot}98} = 4{\cdot}2$ say <u>1 in 4</u>.

Substituting the value of X (*i.e.* 1 in 4 approximately) and taking moments about O as on the previous page, we get.

	V. Force.		Arm.		Moment.
Live and dead loads ..	119·2	×	4·35	=	518·5
Abutment portion .. $= \frac{2{\cdot}08 \times 8{\cdot}31}{2}$	8·6	×	6·40	=	55·0
Abutment below girder seating (2·33 × 9·81) ..	22·9	×	4·54	=	104·0
Ballast wall, surcharge (3·38 × 21·0) and masonry below =	71·0	×	1·19	=	84·5
Tractive or Braking effort effects =	9·6	×	9·81	=	94·2
Earth Pressure =	47·3	×	6·11	=	289·0
Total ..	221·7				1145·2

Distance of Resultant from $O = \frac{1145{\cdot}2}{221{\cdot}7} = 5{\cdot}17$ feet.

Width of abutment at bed level=7 ′-9½″ and 2-3rd of this =2× (2′-7⅛″) = 5′-2⅓″ = 5·19 ft *i.e.*

Resultant falls just in middle-third.

$\therefore$ eccentricity =1/6 width of base.

Maximum compression at bed level in abutment masonry

$$= \frac{W}{A} \times 1 \pm 6 \times \frac{(\text{eccentricity})}{(\text{base width})} = \frac{W}{A} \times 2 = \frac{2W}{A}$$

$$= \frac{2 \times 221 \cdot 7}{20 \times 7 \cdot 79} = \frac{221 \cdot 7}{77 \cdot 9} = 2 \cdot 72 \text{ tons per sq. foot.}$$

Section of Bridge showing Founds.

Live and dead load at founds level will be distributed over the whole length of founds which is equal to 22 feet.

Therefore load on one foot length of founds=

$$\frac{119 \cdot 2 \times 20}{22} = \frac{1192}{11} = 108.4 \text{ Cwts.}$$

Foundation Pressures :—

For reference please see figure No. 1 on Plate VIII.

Take moments about 0 ′

H. Force.		V. Force.	Arm.	Moment.
Live and dead loads	..	108·4×	5·35 =	575·0
Abutment portion $\frac{2 \cdot 08 \times 8 \cdot 31}{2}$	..	8·6×	7·40 =	63·6
Abutment below girder seating (2·33×9·81)	.. ..	22·9×	5·54 =	126·9
Ballast wall, surcharge and masonry below	3·38×21·0=	71·0×	2·19 =	155·5
Founds 10.75×6	..	64·5×	5·38 =	347·0
Back filling 1×21·0	..	21·0×	0·50 =	10·5
Earth Pressure 47·3	..	×	6·11 =	289·0
Braking or Tractive effort effects=9·6	..	×	9·81 =	94·2
Total	..	296·4		1661·7

Distance of Resultant 0 ′ $= \frac{1661 \cdot 7}{296 \cdot 4} = 5 \cdot 60$ feet.

Eccentricity=5·60—5·38=·22

$\therefore$ Maximum Pressure on founds. $= \frac{W}{A}\left(1 \pm \frac{6 \times \text{eccentricity}}{\text{base width}}\right)$

$$= \frac{296 \cdot 4}{20 \times 10 \cdot 75}\left(1 \pm \frac{6 \times \cdot 22}{10 \cdot 75}\right) = \frac{296 \cdot 4}{215}\left(1 \pm \frac{1 \cdot 32}{10 \cdot 75}\right) = 1 \cdot 38 \times 1 \cdot 12$$

=1.55 Tons per square foot.

Depth of founds, as per Rankine's formula $= \frac{P}{W}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2$

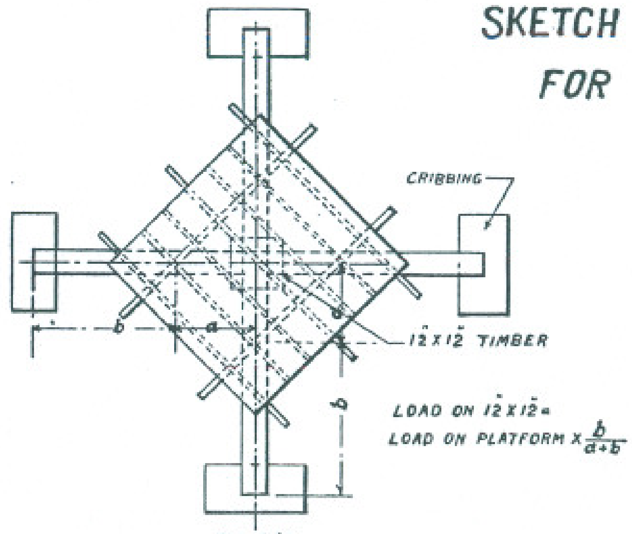
Here $P = 1 \cdot 55$, $W = 1/20$. $\phi = 30°$.

$$\therefore \text{depth of founds.} = \frac{1 \cdot 55}{1/20} \times (1/3)^2 = \frac{1 \cdot 55 \times 20}{9} = \frac{31 \cdot 00}{9}$$

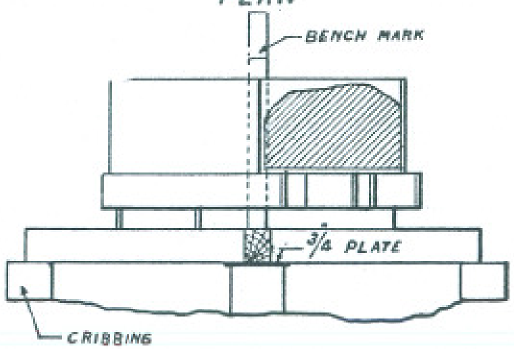
$$= 3 \cdot 44 \text{ ft.}$$

But in the design it is provided as 6 feet which is quite ample.

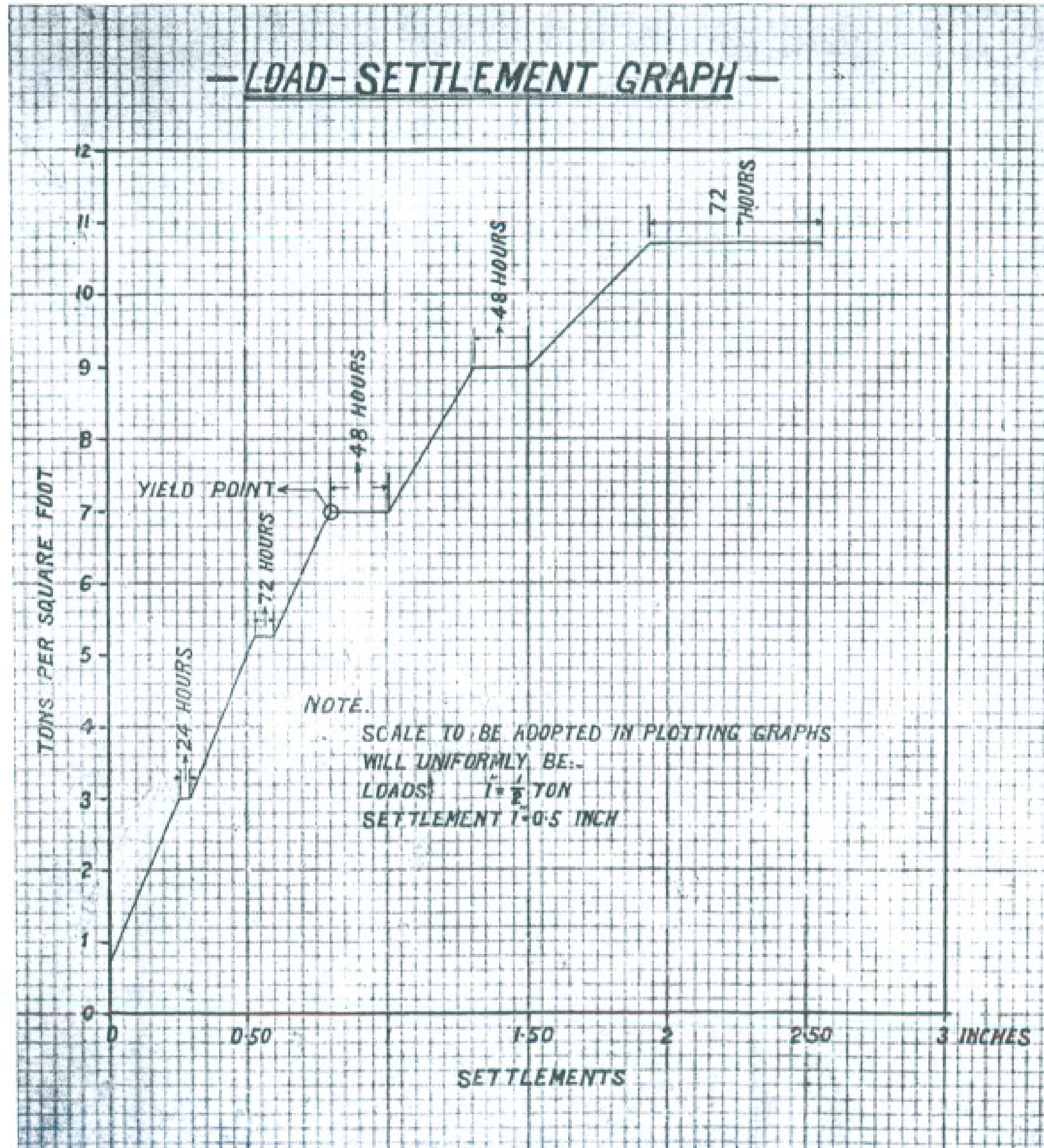
SKETCH SHOWING ARRANGEMENTS FOR SOIL PRESSURE TESTS



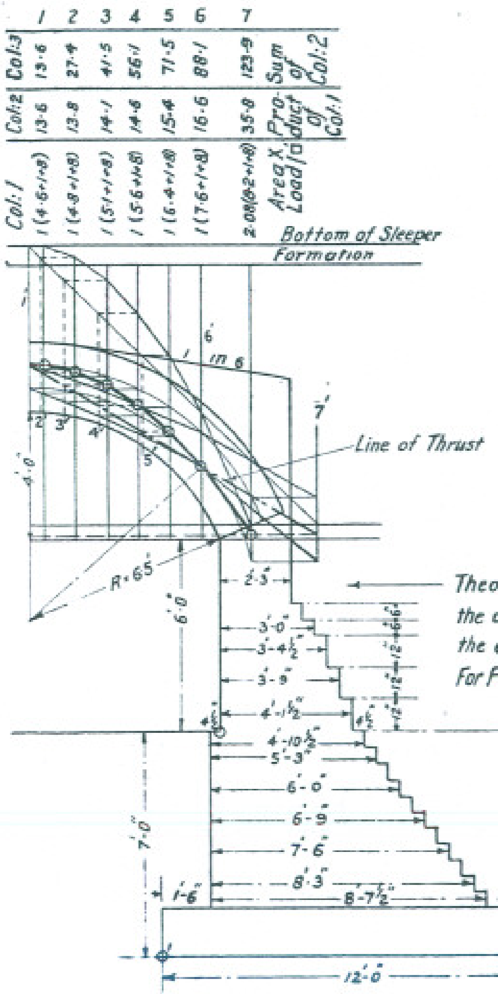
PLAN

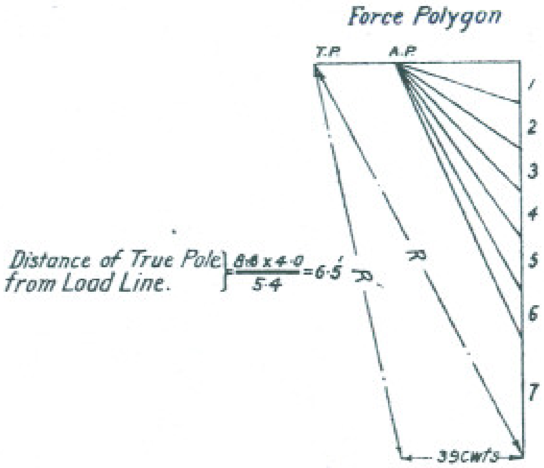


ELEVATION



-LOAD-SETTLEMENT GRAPH-
TONS PER SQUARE FOOT
0
1
2
3
4
5
6
7
8
9
10
11
12
24 HOURS
72 HOURS
YIELD POINT
48 HOURS
48 HOURS
72 HOURS
NOTE.
SCALE TO BE ADOPTED IN PLOTTING GRAPHS
WILL UNIFORMLY BE:-
LOADS 1" = ½ TON
SETTLEMENT 1" = 0·5 INCH
0
0·50
1
1·50
2
2·50
3 INCHES
SETTLEMENTS





DESIGN OF A 12 FT. ARCH BRIDGE

Scales { Load 1" = 40 cwts.
Linear 1" = 4 ft.

H. M. LOADING

PRESSURE IN ARCH RING ... 4·84 Tons/□'
" AT BED LEVEL ... 3·75 "
" " FOUNDS ... 1·5 "

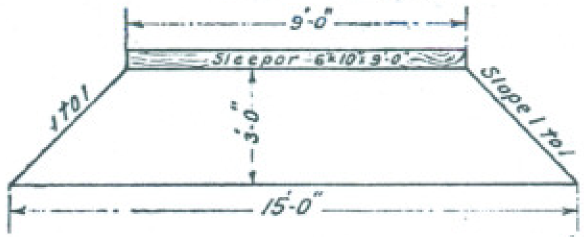
(1) Line of least resistance is drawn by Fuller's method.
(2) Live load & impact are taken up to the ends of skew backs.
(3) The rest of abutments carry dead loads only.
(4) The weight of masonry & earth is taken as one cwt/
(5) One ft. length of arch & abutment is considered.
(6) Arch is fully loaded.

DATA & CALCULATIONS

STANDARD OF LOADING = H.M.

I. 1 Clear span ... 12 ft.
2 Rise of arch ... $\frac{\text{Span}}{3}$ ... 4 ft.
3 Radius ... $\frac{13}{24}$ Span ... 6·5 ft.
4 From several examples of arch designs worked out on this Railway it has been found that the thickness of a masonry arch of a segmental shape for H.M. loading is approximately ·91$\sqrt{R}$. vide Plates Nos III, IV, V, VI & VII
5 Overall length ... 16·16 ft.
6 Cushion up to bottom of sleepers ... 3 ft.
7 Height of springing from floor level ... 6 ft.

ii **Load per Sq: ft. on the Arch**



Axle load ______________________________ 28 Tons

Distance the axles are apart ______________ 5.6 ft.

$\therefore$ Load per foot length _______ $28 \div 5.6$ _______ 5 Tons

Area over which this load is distributed $15 \times 1 = 15$ Sq. ft.

$\therefore$ Load / sq. ft. ___ $\frac{5 \times 20}{15} = \frac{20}{3}$ ____________ 6.67 Cwts.

Add 20% for impact ___ $\frac{20}{3} \times \frac{1}{5} = \frac{4}{3}$ ______ 1.33 "

Total __ 8.00 Cwts

iii Value of Earth pressure & its point of application.

Earth pressure (As per Rankine's Formula) $= \frac{1}{2} WH [H+2h] \left[\frac{1-\sin\phi}{1+\sin\phi}\right]$

Where H Distance from floor level to crown of arch $= 12.25$ ft.

h = Distance from crown of arch to top of surcharge $= 3.33$ ft.

ϕ = angle of repose ________________________ 30°

Earth pressure ___ $= \frac{1}{2} \times 1 \times 12.25 \left[12.25 + 2 \times 3.33\right] \frac{1-\frac{1}{2}}{1+\frac{1}{2}}$

$$\frac{12.25 \times 18.91}{6} = \frac{231.6}{6} = 38.6 \text{ Say 39 cwts.}$$

Point of application of earth pressure above floor level ____ $\frac{H}{3}\left[\frac{H+3h}{H+2h}\right] = \frac{12.25}{3}\left[\frac{12.25+3 \times 3.33}{12.25+2 \times 3.33}\right]$

$$= \frac{12.25 \times 22.24}{3 \times 18.91} = \frac{272.44}{56.73} = 4.8 \text{ ft.}$$

IV Thickness of Abutment at Floor Level.

From the force polygon the resultant thrust R works out to be 140 cwts Combining this with the horizontal earth pressure as shown in the figure.

We get the resultant R intersecting the base at floor level at a distance of 2.68 ft. from O as worked below:

Let x be the width of base

Take moments about O

	Load		Arm		Moments
	124.00	×	2.68	=	332.32
6.5 x 2.08 =	13.52	×	1.04	=	14.06
$15.58[x - 2.08] =$	$15.58x - 32.41$	×	$\frac{x+2.08}{2}$	=	$7.79x^2 - 33.71$
Total	$15.58x + 105.11$				$7.79x^2 + 312.67$

But the resultant must pass at the exteme edge of the middle third.

$\therefore$ The distance of the resultant from O $= \frac{7.79x^2 + 312.6}{15.58x + 105.1}$

ie: $23.37x^2 + 938.01 = 31.16x^2 + 210.22x$

ie: $7.79x^2 + 210.22x - 938.01 = 0$

Solving the quadratic equation the value of $x = 3.9$

So the thickness in the nearest $4\frac{1}{2}$" bricksize works out to 4.12 ft.

Actual Pressure at Floor level.

Substituting the value of x ie: 4.12 in $\frac{7.79x^2 + 312.6}{15.58x + 105}$ we get the distance of resultant from O

$$= \frac{312.67 + 132.23}{105.11 + 64.19} = \frac{444.9}{169.3} = 2.63 \text{ f}$$

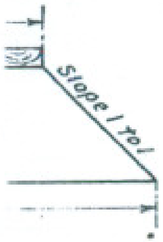
The total vertical load ______________ 169.3 Cwt

$\therefore$ Pressure at floor level ____ $\frac{W}{A}\left[1 \pm \frac{6e}{\text{base width}}\right]$

e = eccentricity $= 2.63 - \frac{4.12}{2} = 2.63 - 2.06 = .57$

A ________________ 4.12×1 __ __ $= 4.1$

e Arch



--------- 28 Tons

--------- 5·6 ft

·6 -------- 5 Tons

uted $15 \times 1 = 15$ Sq. ft

------ 6.67 Cwts.

------ 1·33 "

Total -- 8·00 Cwts

of application.

$-WH\left[H+2h\right]\left[\frac{1-\sin\phi}{1+\sin\phi}\right]$

own of arch = 12·25 ft

p of surcharge = 3·33 ft

---------- 30°

$2\cdot25+2\times3\cdot33\Big]\frac{1-\frac{1}{2}}{1+\frac{1}{2}}$

6 = 38·6 Say 39 cwts.

above

$\frac{25+3\times3\cdot33}{5+2\times3\cdot33}\Big]$

$\frac{2\cdot44}{6\cdot73} = 4\cdot8$ ft

IV Thickness of Abutment at Floor Level.

From the force polygon the resultant thrust R works out to be 140 cwts Combining this with the horizontal earth pressure as shown in the figure.

We get the resultant R intersecting the base at floor level at a distance of 2·68 ft from O as worked below:

Let x be the width of base

Take moments about O

	Load	Arm	Moments
	124·00	× 2·68 =	332·32
$6\cdot5 \times 2\cdot08 =$	13·52	× 1·04 =	14·06
$15\cdot58[x-2\cdot08] =$	$15\cdot58x - 32\cdot41$	$\times \frac{x+2\cdot08}{2} =$	$7\cdot79x^2 - 33\cdot71$
Total	$15\cdot58x + 105\cdot11$		$7\cdot79x^2 + 312\cdot67$

But the resultant must pass at the extreme edge of the middle third.

$\therefore$ The distance of the resultant from $O = \frac{7\cdot79x^2+312\cdot67}{15\cdot58x+105\cdot11} = \frac{2x}{3}$

ie: $23\cdot37x^2 + 938\cdot01 = 31\cdot16x^2 + 210\cdot22x$

ie: $7\cdot79x^2 + 210\cdot22x - 938\cdot01 = 0$

Solving the quadratic equation the value of $x = 3\cdot9$ ft

So the thickness in the nearest 4½" bricksize works out to 4·12 ft.

Actual Pressure at Floor level.

Substituting the value of x ie: 4·12 in $\frac{7\cdot79x^2+312\cdot67}{15\cdot58x+105\cdot11}$ we get the distance of resultant from O

$$= \frac{312\cdot67+132\cdot23}{105\cdot11+64\cdot19} = \frac{444\cdot9}{169\cdot3} = 2\cdot63 \text{ ft}$$

The total vertical load -------------- 169·3 Cwts.

$\therefore$ Pressure at floor level --- $\frac{W}{A}\left[1 \pm \frac{6e}{\text{base width}}\right]$

e = eccentricity $= 2\cdot63 - \frac{4\cdot12}{2} = 2\cdot63 - 2\cdot06 = \cdot57$

A ---------------- 4·12×1 --- --- = 4·12

$\therefore$ Pressure at floor level $\frac{169\cdot3}{20\times4\cdot12}\left[1 \pm \frac{6\times\cdot57}{4\cdot12}\right]$

$$\frac{169\cdot3}{82\cdot4}\left[1+\frac{3\cdot42}{4\cdot12}\right] = 2\cdot05 \times 1\cdot83 = 3\cdot75 \text{ Tons}/\square'$$

VI Founds.

Take moments about O'

	Arm	moments
	$169\cdot30 \times 4\cdot51$	763·54
wt. of founds $12\times7 = $	$84\cdot00 \times 6\cdot00$	504·00
wt of back fill. $15\cdot58\times6 =$	$93\cdot48 \times 9\cdot00$	841·32
	346·78	2108·86

Distance of resultant from $O' = \frac{2108\cdot86}{346\cdot78} = 6.08$

e = eccentricity $= 6\cdot08 - \frac{12\cdot0}{2} = 6\cdot08 - 6\cdot00 = \cdot08$

$\therefore$ Soil pressure $= \frac{W}{A}\left[1 \pm \frac{6e}{\text{base width}}\right] = \frac{346\cdot78}{20\times12}\left[1 \pm \frac{6\times\cdot08}{12}\right]$

$1\cdot44 \times 1\cdot04 = 1\cdot5$ Tons$/\square'$

VII Depth of Founds by Rankine's Formula

$$= \frac{P}{W}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$

Here $P = 1\cdot5$ Tons.

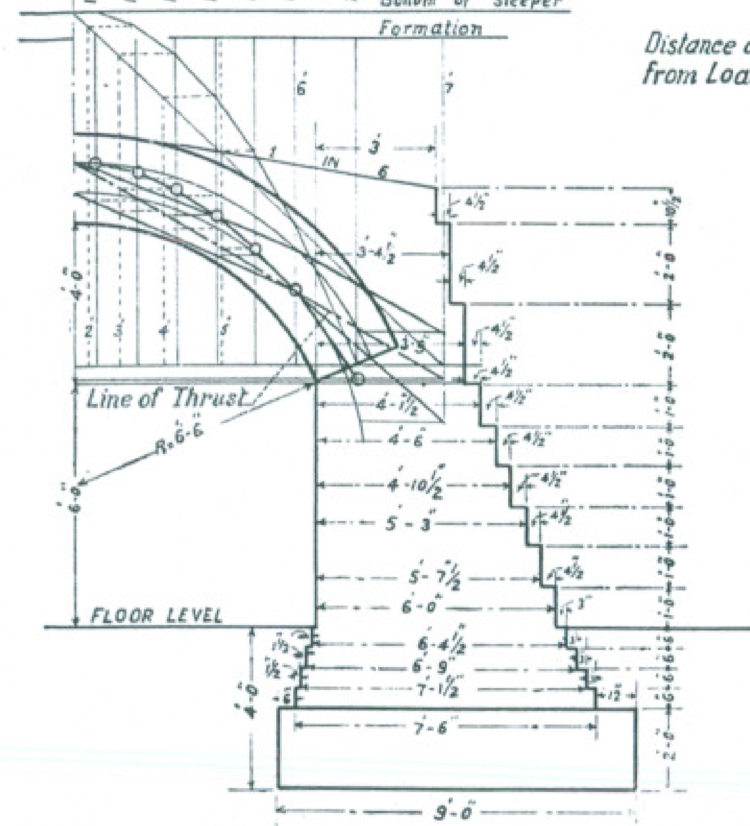
$W = 1$ Cwt. $= \cdot05$ Tons.

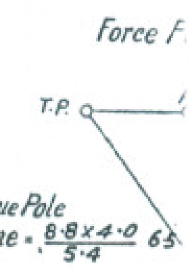
$\phi = 30^\circ$

$$\text{Depth} = \frac{1\cdot5}{\cdot05}\left[\frac{1-\frac{1}{2}}{1+\frac{1}{2}}\right]^2 = 30 \times \left(\frac{1}{3}\right)^2 = \frac{30}{9} = 3\cdot33 \text{ ft}$$

against 7 ft provided in the design.

	Col: 1	Col 2	Col: 3
1	1(4·6+1+8)	13·6	13·6
2	1(4·8+1+8)	13·8	27·4
3	1(5·1+1+8)	14·1	41·5
4	1(5·6+1+8)	14·6	56·1
5	1(6·4+1+8)	15·4	71·5
6	1(7·6+1+8)	16·6	88·1
7	2·08(8·2+1+8)	35·8	123·9
	Area x Load/□	Product of Col:1	Sum of Col:2





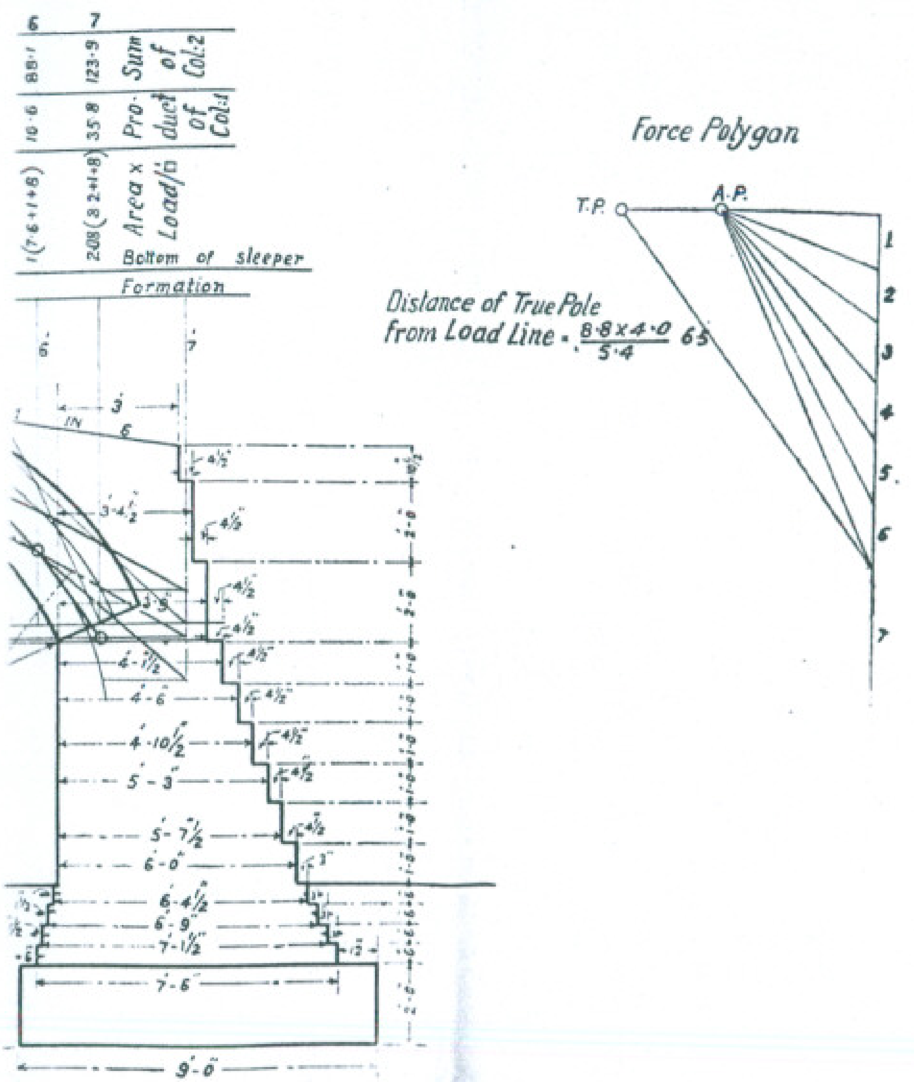
Distance of True Pole from Load Line = $\frac{8 \cdot 8 \times 4 \cdot 0}{5 \cdot 4}$ 6·5

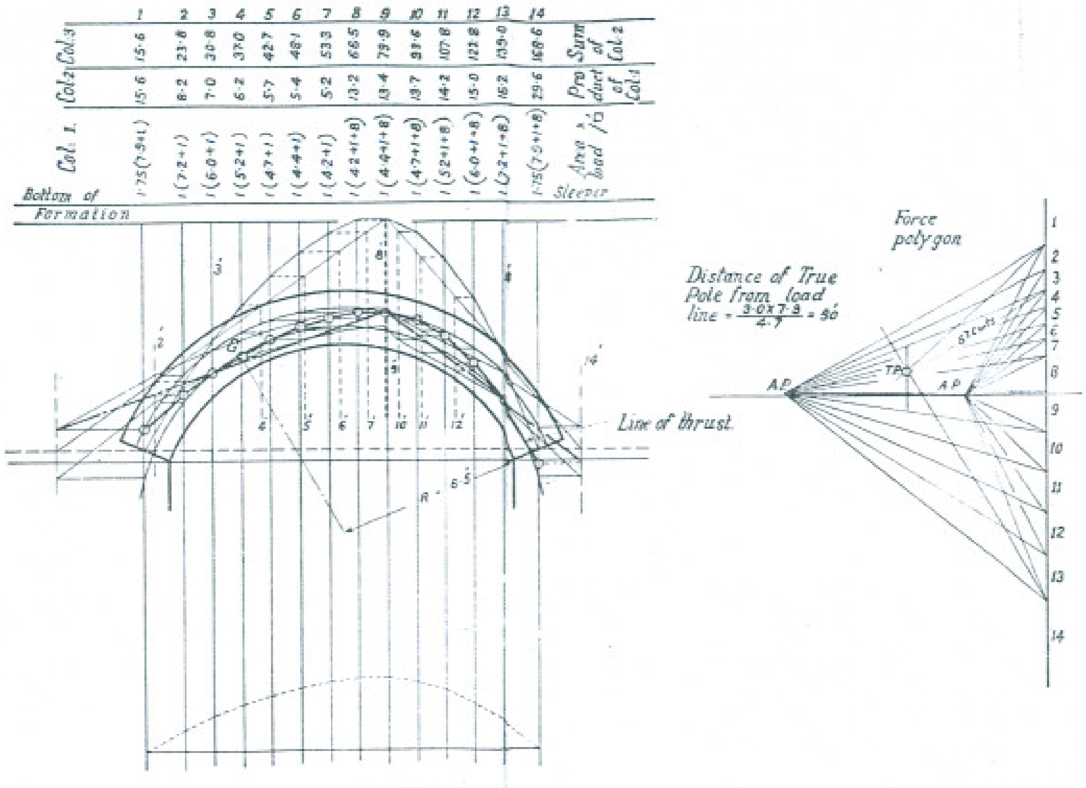
DESIGN OF A 12 F$^{\text{T}}$ ARCH BRIDGE

Scales {Load 1" = 40 cwts / Linear 1" = 4 ft

PRESSURE IN ARCH RING 4·84 Tons /□'
" AT BED LEVEL 1·67 " "
" " FOUNDS 1·50 " "

Section of Abutment applied in actual practice in preference to theoretical Section worked in Plate N° III





HALF LOADED

DESIGN OF A 12 FT. ARCH

DETERMINATION OF LINE OF THRUST

(FULLER'S METHOD)

STANDARD OF LOADING = H.M.

Thickness - $.9\sqrt{Rcd}$ - $4\frac{?}{2}$

Span .... = 12 ft.

Rise .... = 4 ft.

Scale { Load ..... 1" = 40 cwts
Linear 1" = 4 ft.

Compression at 6 = 4.27 Tons/□'

Tension " " .71 Tons/□'

{ The arch ring develops tension which is to be avoided. See Plate No VI

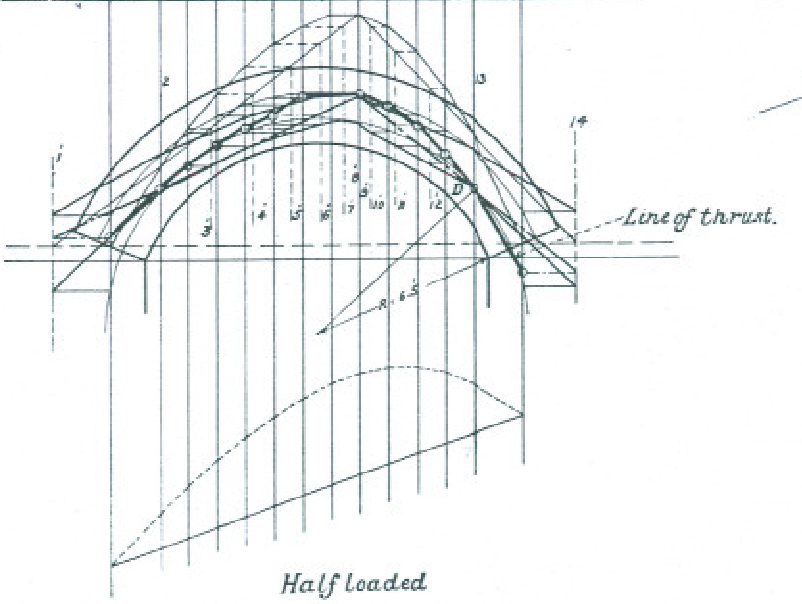
DESIGN OF A 12 FT. ARCH
DETERMINATION OF LINE OF THRUST
(FULLER'S METHOD)

STANDARD OF LOADING = H.M.
THICKNESS .... = ·31 √RAD: + 4½
SPAN .... = 12 FT.
RISE .... = 4 FT.
SCALES { LOAD 1" .... = 40 CWTS.
LINEAR 1" .... = 4 FT.
{ PRESSURE AT D = 3.63 TONS/□' WHICH IS RATHER LOW FOR BRICK WORK.

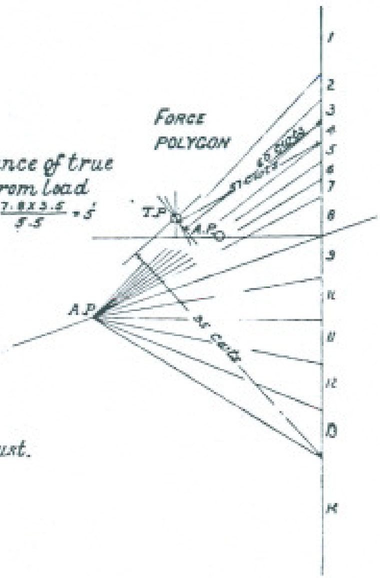
	Col. 1. Area × load/□'	Col. 2 Product of Col. 1.	Col. 3 Sum of Col. 2
1	2·4(8·4+1)	22·6	22·6
2	1(8·0+1)	9·0	31·6
3	1(6·8+1)	7·8	39·4
4	1(6·0+1)	7·0	46·4
5	1(5·5+1)	6·5	52·9
6	1(5·2+1)	6·2	59·1
7	1(5·0+1)	6·0	65·1
8	1(5·0+1+8)	14·0	79·1
9	1(5·2+1+8)	14·2	93·3
10	1(5·5+1+8)	14·5	107·8
11	1(6·0+1+8)	15·0	122·8
12	1(6·8+1+8)	15·8	138·6
13	1(8·0+1+8)	17·0	155·6
14	2·4(8·4+1+8)	41·8	197·4

Bottom of Sleeper

Formation



Half loaded



DESIGN OF A 12 FT. ARCH
DETERMINATION OF LINE OF THRUST
(FULLER'S METHOD)

STANDARD OF LOADING = H-M.

Thickness = ·31√Rad.
Span = 12 ft.
Rise = 4 ft.

Scales {Load 1" = 40 cwts.
{Linear 1" = 4 ft.

Pressure at A = 4.22 Tons/□'
{This is the correct thickness for practical purposes.

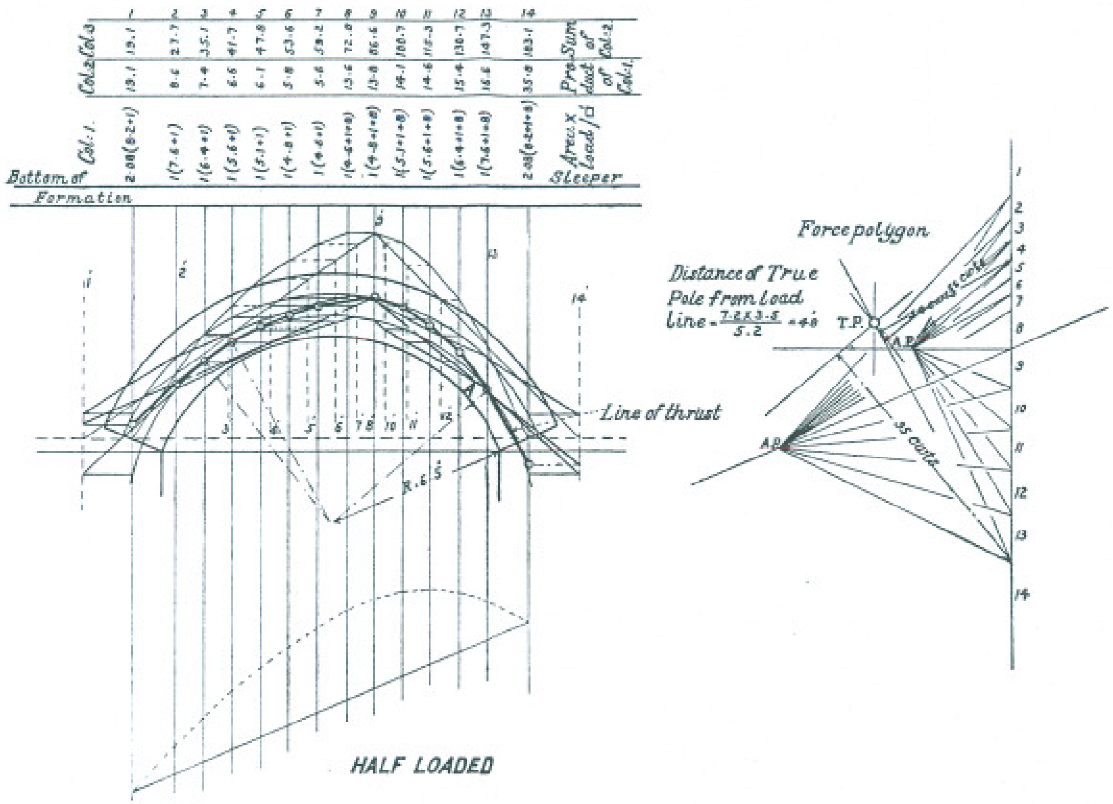
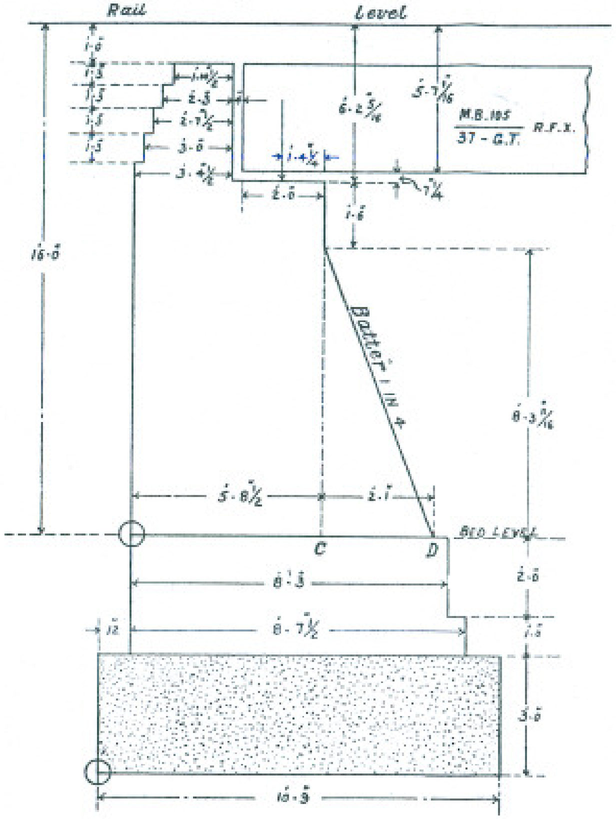


FIG. 1



DESIGN OF ABUTMENT FOR 40FT. GIRDER BRIDGE UNDER H.M. LOADING

5'.6" Gauge.

Single Line

Clear Span = 40 ft.

Overall Length = 44ft.

Effective Span = 42'-8½ (Stepped bearings have been used)

FIG. 2

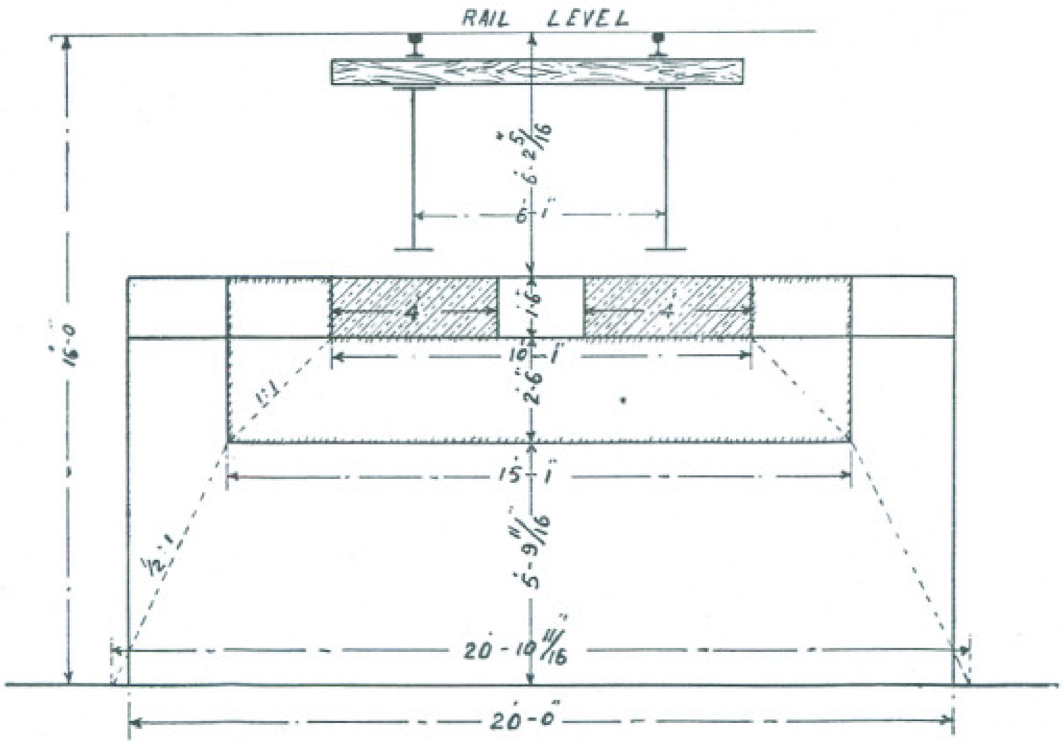
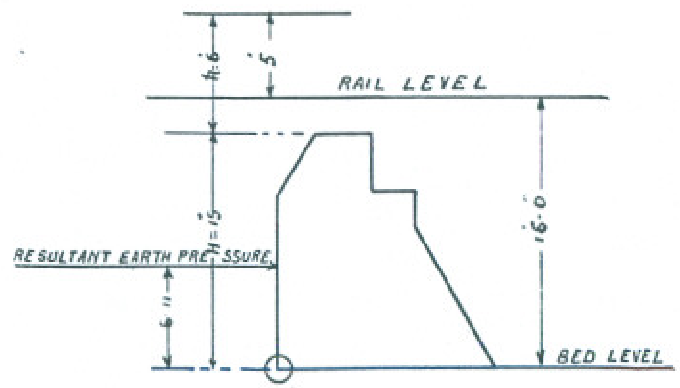

RAIL LEVEL
16'-0
6'-2 5/16"
6'-1"
1'-6"
4'
4'
10'-1"
2'-6"
1:1
15'-1"
5'-9 11/16"
½:1
20'-10 11/16"
20'-0"

FIG. 3


4'-6
5'
RAIL LEVEL
H=15
16'-0
RESULTANT EARTH PRESSURE
5'
BED LEVEL

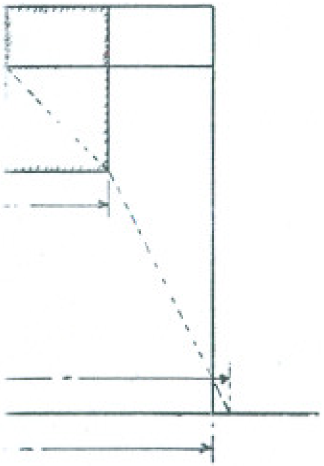


FIG. 3

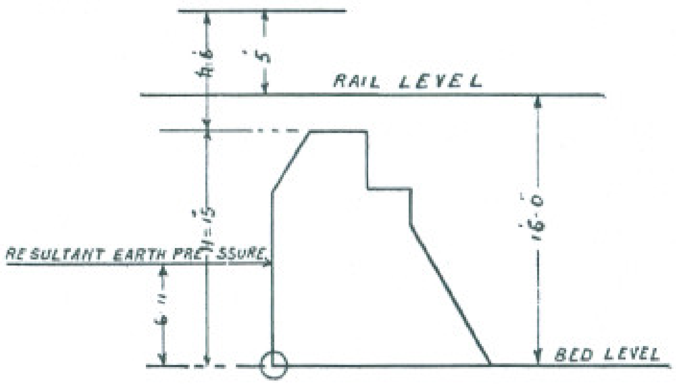
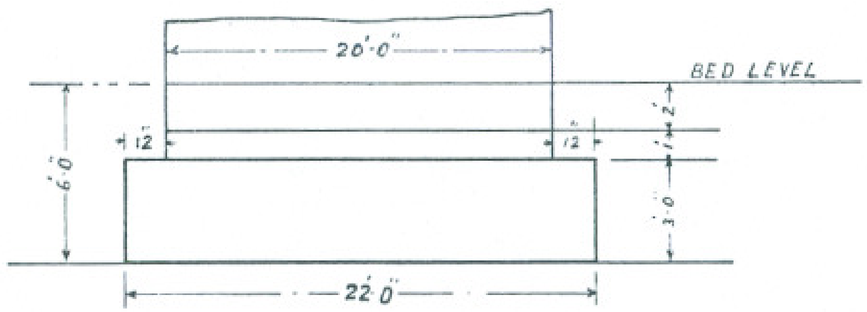


FIG. 4

L. SECTION OF BRIDGE SHOWING FOUNDS.



DISCUSSION.

The **Author** introduced his paper and remarked that he did not pretend to have discovered any new methods of calculations and analysis. During the last four years, it fell to his lot to design numerous new structures and analyse old ones and he found certain difficulties in doing so. These had been summarised on page 142 and it was hoped a discussion on them would prove useful.

He was inclined to think that critical attention was not paid often enough in this country to the design of masonry structures and their foundations. In this connection he could recall an incident which had its own moral. In connection with a meeting of the Institution of Engineers (India) they were being shown round the works at New Delhi. When visiting one of the buildings, the writer asked the engineer-in-charge as to the foundation pressures for which the building had been designed. The reply was something less than one ton per square foot. On being asked whether it was not too low an assumption for the soil which was more like soft rock than clay, his reply was " You see we cannot take risks in a building like this where such and such a high official has to reside." Quite so, they must not take risks in building not only residential quarters for high officials but even in building menial quarters, but at the same time economy must form a necessary function of design. The object of the paper was to stimulate interest in the performance of local experiments on the bearing capacities of soils and masonry before proceeding to design structures.

On page 141, mention was made of Fuller's graphical method of drawing the line of thrust. The author had not come across a strict geometrical proof of this method and would be obliged if any one would refer him to a text book where this was given.

It was regretted that there were a number of printing and other minor mistakes in the calculations given in Appendix III page 149. These however did not materially affect the results. The following were the corrections to be made :—

Page.	*Line.*	*Errors.*	*Corrections.*
141	21	line.	lime.
150	15	inch.	inches.
	26	Abutment portion.	Abutment △ portion.
	31	1·19	1·69
151	3	415 X^2	4·15 X^2
	10	Batter changes from 4·2 to 3·63 but is kept as 1 in 4.	
	14	Abutment portion.	Abutment △ portion.
	20	9·6	(9·6)
	21	47·3	(47·3)
152	14	Abutment portion.	Abutment △ portion.
	26	Resultant 0′	Resultant from.
	30	Pressure changes from	1·55 tons to 1·6 tons.

Page.	*Line.*	*Errors.*	*Corrections.*
Plate III.		Stanbard.	Standard.
iii, line 2		$\frac{1}{2}$ WH [H+2h] $\left[\frac{1-\text{Sin } \phi}{1+\text{Sin } \phi}\right]$	$\frac{1}{2}$ WH [H+2h] $\left[\frac{1-\text{Sin. } \phi}{1+\text{ Sin. } \phi}\right]$
line 3.		H Distance	H=Distance.
iv, line 5.		R	R′
Plate IV.		The heading Plate IV is to be written.	
Plate VIII.		$\frac{\text{M. B. 105.}}{\text{37-G. T.}}$ R.F.X	$\frac{\text{M. B. 105.}}{\text{37-G. T.}}$ P.F.X.

Mr. W. T. Everall congratulated Mr. Dhawan on his interesting paper, dealing with the important subject of methods of calculation and analysis of certain masonry and concrete structures.

The speaker was particularly interested in the question of longitudinal forces due to tractive effort and breaking effect, referred to on page 143, and to what extent they were transmitted at the girder seating level to the abutment or pier. He particularly mentioned the following few remarks for the information of the members :—

Since this paper was written, some experimental work, at the instigation of Mr. Pavry, had been carried out by the bridge staff of the N.-W. Railway with the object of determining to what extent the tractive forces were transmitted under various conditions, to the girder supports. Tests were made on a bridge consisting of 40 ft. open deck spans.

One of the spans was isolated by disconnecting the track and wheel guard connections, and the girders mounted on rollers at each end and connected longitudinally to the masonry by a tension link. Fereday's Optical Stress Recorders attached to the link were used to record the forces induced during the passage of the test train.

The first series of runs were made with the fish plates of the running rails disconnected at the approaches of the test span so that the whole of the horizontal tractive force, amounting to 23 tons, exerted by an X. G. Engine on the span, was recorded by the Stress Recorders, excluding the small percentage transmitted to the masonry by virtue of the friction of the roller bearings.

The following combinations were tested and the results recorded :—

(i) With the joints of the running rails fished up, but guard rails disconnected—79 per cent. of the horizontal force passed into the ballasted approach at the end of the bridge.

(ii) With the joints of the outside guard rails fished up, but the running rails disconnected—50 per cent. of the horizontal force was transmitted into the ballast of the approach.

(iii) With the joints of the running rails and also the guard rails all connected up—82 per cent. of the horizontal force was dissipated.

In a ballasted floor span under similar tests, it was found that with the rail joints disconnected and the ballast continuous, a relief of nearly 70 per cent. was obtained.

From these results, which confirmed previous tests made on other bridges, it would be seen that a very substantial relief to the abutment or pier support, was afforded by the permanent-way ; and the provision of continuous guard rails further ensured relief, in the event of the running rail connections being loose or not functioning to their fullest extent.

The piers and abutments, of the shorter spans, below 60 ft., due to their more slender dimensions, were usually more influenced by these horizontal forces than were the piers and abutments of larger spans, and it was an advantage that short spans were usually provided with flat sliding bearings, as these tended to distribute the forces on to both supports, whereas roller bearings tended to concentrate them on the fixed ends only.

The speaker said that it would be of interest to mention that last year he discussed this problem with Dr. Buhler, the Chief Bridge Engineer of the Swiss Federal Railways during a visit he made to Switzerland to inspect some railway bridge works.

Dr. Buhler told him that it was their practice to utilize the permanent-way for dissipating the horizontal forces on bridges.

In the shorter spans, no rail joint was permitted on or near the span, this was to some extent simplified by the use of rails 60 to 70 ft. long. In many of the larger bridges, he noticed that the rail joints had been dispensed with by electrically welding the rails together.

The Swiss Railways, under these conditions, legislated for only 30 to 50 per cent. of the horizontal forces at the running rails, being applied at the supports.

The British Engineering Standards Association, in their Panel Committee, dealing with bridge girder loads and stresses (on which the speaker acted as a member last year) had also brought the question of horizontal forces under review, and had appointed a sub-committee to investigate the position with a view to ascertaining the degree of relaxation which might be applied, when using their present formulæ.

One member of the Committee reported that tests had been made of fished 75 lbs. rail joints with the bolts normally tightened, as in the track, and these were found to carry loads of 12½ tons on each rail, without any movement or slip.

The speaker remarked that they were proceeding with their investigations on the N.-W. Railway, and hoped shortly to clear up the problem after determining the longitudinal forces in the piers of other bridges, both of open and ballasted types.

In the light of their experiments, the speaker was decidedly in favour of an immediate relaxation of 50 per cent. when using the Government of India or the B. E. S. A. Formulæ for piers and abutments of girder bridges with open floors, and something even more in the case of ballasted floors.

The President remarked that as the discussion on this paper had been very short and as Mr. Dhawan had nothing to say in reply to Mr. Everall's observations, perhaps he had better explain the genesis of this paper. About 1924-25, the Railway Board laid down heavier standards of axle loading for the future. At present, the axle loads were somewhere between 16 and 17 tons, and in future they proposed to run axle loads of 20 to 22½ and as much as 28 tons. As the 28 tons axle loads might come along during the life of the structures that they were now putting up, masonry bridges were being designed for the full 28 tons axle loads, even though they did not expect 28 tons loads on the N.-W. Railway for many a long year. In the first few structures designed for these axle loads by his design section headed by Mr. Dhawan, the sections looked excessively heavy. Though the speaker was a firm believer in theory, it was necessary to take theory with a certain amount of judgment and commonsense. Thousands of masonry bridges on the Railway had stood perfectly well under 16, 17 and 18 ton axle loads and only a very small proportion failed from time to time. Now if those structures had stood for 50 to 70 years under axle loads rising from 12 to 18 tons, it was unreasonable to put up new structures, in which the increase of section over that in existing structures was out of all proportion to the proposed increase in axle loads.

Mr. Dhawan's paper was a review of the attempt made by the Railway to bridge the gap between theoretical analysis according to current data and methods, and practical experience.

NOTE ON THE PREPARATION OF DIAGRAMS AND DRAWINGS.

The following hints are put forward for the benefit of members preparing papers so that the drawings may appear to the best advantage.

Most drawings prepared for engineering use are totally unsuited for reproduction. Special drawings should be prepared.

All drawings should be in black only, as opaque a black should be used as possible on a white paper or on tracing cloth. The usual Indian ink insufficiently rubbed down makes a clear reproduction difficult. Red is unsuitable and other colours impossible.

Diagrams on the usual departmental squared paper (or any other) are quite unsuitable for reproduction and should be traced.

The appearance of most drawings is enhanced by photographic reduction. To enable the press to do so the following points are brought to notice.

Lines must be boldly drawn of fair thickness so that they may not disappear altogether when photographed. The thickness of line must be regulated according to the amount of reduction intended.

Printing should not only have the size of letters sufficient but the thickness of line in the body of each letter or number must be attended to. A fair proportion of thickness to height may be judged by referring to the standard stencils in any drawing office.

Scales should be actually drawn on the drawing and not indicated by fractional equivalents.

It is suggested that drawings foolscap depth with $\frac{1}{4}''$ stencilled lettering or double foolscap depth with $\frac{1}{2}''$ stencils be used as standards