TECHNICAL MEMORANDUM (BRIDGES) No. B.E.4.

P. Tatlow, C.Eng; M.I.C.E,
Pear Tree Cottage,
Little Tangley, Wonersh,
Guildford, Surrey,

MINISTRY OF TRANSPORT

THE ASSESSMENT OF HIGHWAY BRIDGES FOR CONSTRUCTION AND USE VEHICLES

PART I : BASIC PRINCIPLES

PART II : QUICK ASSESSMENT

PART III : BRICK AND MASONRY ARCHES (~~TO FOLLOW~~)

DATE: JANUARY 1967

Amend 1 30 Oct 1970

THE ASSESSMENT OF HIGHWAY BRIDGES FOR CONSTRUCTION AND USE VEHICLES

CONTENTS

WEIGHTS OF MATERIALS TO BE USED

Aluminium	170 lb./cu.ft.
Cast Iron	450 " " " 720
Steel	490 " " "
Wrought Iron	480 " " "
Brickwork	140 " " " 224[illegible]
Concrete	150 " " " 2400
Masonry	144 " " "
Earth, Sand, Gravel and Miscellaneous Filling	135 " " " 2[illegible]
Macadam	144 " " " 230[illegible]
Timber	40 " " "

ASSESSMENT OF HIGHWAY BRIDGES FOR CONSTRUCTION AND USE VEHICLES

Foreword

This Code has been written by the Ministry of Transport and the British Railways Board in consultation with the County Surveyors Society.

The Code is divided into two parts. Part I gives the basic principles of assessment with the reasons for their adoption. Part II gives a method of assessment for longitudinal and transverse beams.

It is intended that the principles given shall provide a uniform approach to bridge assessment and the method set out in Part II will allow for the quick assessment of bridges without undue loss of accuracy.

PART I BASIC PRINCIPLES

SECTION I GENERAL

101 Scope

The provisions of the Code are directed towards the assessment, for CONSTRUCTION AND USE VEHICLES, of bridges having a capacity less than H.A. loading as specified in B.S.153, Part 3A. These bridges should be considered as substandard and having a limited life and their assessment is subject to the provisos:-

(a) That the replacement of the structure is not unduly delayed.

(b) That the structure is critically examined at such intervals as its condition requires, but at least once in three years.

SECTION 2 DERIVATION OF VEHICLE TRAINS

201 Regulation of Vehicles

Vehicle design is controlled by the Construction and Use Regulations* to which reference should be made for details of maximum permitted loads on wheels, axles, and vehicles. An approximation of the limitations at the date of issue of this Code follows:-

* The Motor Vehicles (Construction and Use) Regulations 1963 as amended.

201 Maximum weight of 1 wheel with twin tyres or low profile

tyre (Not less than 12 in. wide)	5 tons
" " " 1 axle with two wheels as above	10 tons
" " " 1 axle with four wheels	11 tons
" " " 2 axles not less than 4 ft. apart	18 tons
Maximum gross weight of vehicles with 2 axles and no minimum wheelbase specified.	14 tons
Maximum gross weight of vehicles with 3 axles and no minimum wheelbase specified	20 tons
Maximum gross weight of vehicles with 4 axles and no minimum wheelbase specified	24 tons

The Regulations control the effects on bridge decks caused by heavy vehicles, by relating their gross weights to minimum axle spacing; thus the increased weights up to 16 tons on two axled vehicles, 28 tons on 4 axled rigid vehicles and 32 tons on articulated vehicles, which were allowed in 1964, are accompanied by minimum axle spacings which ensure that the increase in effect of these heavier vehicles is only marginal.

202 Rationalised Vehicle Trains

(a) Longitudinal Effects

(i) Bending

It has been found possible to cover the longitudinal bending effects produced by all vehicles complying with Construction and Use Regulations by means of two trains.

The first consists of five two axle vehicles with the axle spacing and weight distribution shown in Fig.1(a).

With this train, although the maximum permitted gross weight W of a vehicle with such axle spacing under C. and U. Regulations may not be more than 14 tons, its effects are as great as those of any other permitted vehicle having a gross vehicle weight of up to 20 tons. Any bridge which can carry 14 ton vehicles will be capable of carrying any permitted vehicle up to 20 tons and therefore there will be no assessments greater than 13 and less than 20 tons.

202 The second train consists of three four axle vehicles with the axle spacing and weight distribution shown in Fig.1(b). The effects of 24 ton, 4 axle vehicles on a bridge are greater than that of all other vehicles complying with the Construction and Use Regulation current at the date of issue of this Code except for those with an 11 ton axle or two 9 ton axles 4'6" apart which have a slightly greater effect in the lower spans. In graph 5 the values for the 24 ton vehicle have been adjusted to take account of these effects.

(ii) Shear

Because the restriction will be in terms of axle weight (see Part I Clause 406) and any axle of weight up to 9 tons can be closely followed by another axle of equal weight, the train of vehicles given in Fig.1(b) shall be applied to longitudinal members longer than 7'0". If these members can carry the two nine ton and their associated axles, they can also carry the eleven ton and its associated axles. Members shorter than 7'0" should also be checked for the 11 ton axle before they can be cleared for no restriction in shear.

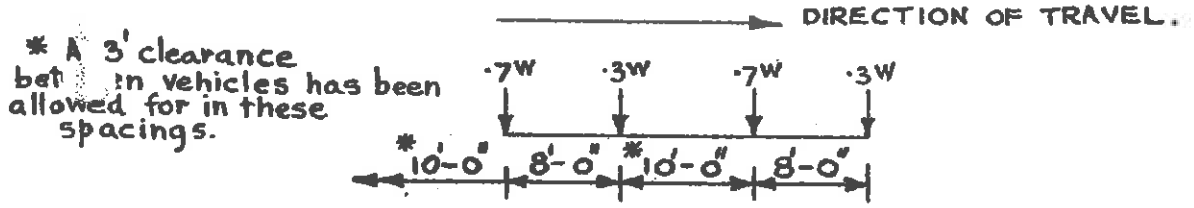


FIG.1.(a) 2 AXLE VEHICLES, W = GROSS WEIGHT OF VEHICLE.

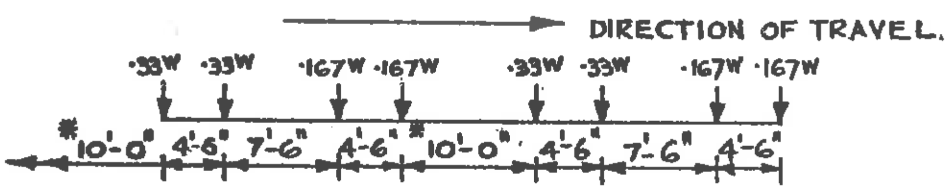


FIG.1.(b) 4 AXLE VEHICLES, W = GROSS WEIGHT OF VEHICLE.

(b) Transverse Effects

Though a deck may carry a certain loading in longitudinal bending, axle loading may also be a criterion where there are transverse members. Within the C. and U. Regulations any axle

202 with a weight up to 9 tons may be followed closely by a further axle of the same weight.

Where allowance can be made for distribution effects (see Clause 303(c)(i)), the two 9 ton axles will have a greater effect on cross girders than will a single 11 ton axle and it is only when distribution is not permissible, that the latter need be considered. With this exception any transverse member capable of carrying the two 9 ton axles with the associated axles derived from Fig.1(b) will be able to carry the single 11 ton axle with its associated axles permitted by the Construction and Use Regulations current at the date of issue of this Code, and the train of vehicles given in fig.1(b) shall be used when assessing the strength of such a member.

(e) Public Service Vehicles

Public Service Vehicles do not conform to the trains shown in Figs. 1(a) and 1(b), and where a bridge subject to a weight restriction carries a bus route advice is given in Clause 402 and Table 2.

SECTION 3 ASSESSMENT PROCEDURE

301 General

(a) Bridges for which No Assessment is Needed

It may be assumed that the design of bridges built subsequently to 1922 to carry classified or trunk roads at the time of their construction will accommodate H.A. loading and unless they are in bad condition no assessment for Construction and Use traffic will be necessary.

(b) Examination of Structure

The structure shall be examined for possible faults, e.g. corrosion, settlement, faulty material, and allowance made for its condition when the carrying capacity is assessed.

(c) Capacity of Constituent Parts

In arriving at an assessment of the bridge, the foundations, substructure and superstructure should all be considered. When

~~BE 21/2/01~~ BE 21/2/04

The Assessment of Highway Bridges For Construction and Use Vehicles Amendments to Part I

1) Para. 202(a)(i), page, 7, final sentence:-

"In graph 1 .... " should be altered to "In graph 5 .... "

2) Para. 305(b)(ii), page 17:-

After "..... provided:- The girders are known to be firmly embedded in well consolidated filling material", the following sentence should be inserted:-

(except where such material is pure sand or pure clay).

A. D. Holland

A. D. Holland

16 January, 1967

301 dealing with the superstructure, the constituent parts should all be investigated for the loading they carry, e.g. floor plates under wheel loads, cross girders under axle loads, and longitudinal girders under vehicle loads. When a bridge assessment shows that there are wide differences between the strengths of different members, and it appears that strengthening one or more is practicable and worthwhile a note of their various capacities should be made on the calculation sheet.

302 Loading

(a) Application of Vehicle Trains

For longitudinal bending the vehicle trains given in Figs. 1(a) and 1(b) shall be applied as appropriate, see 202 (a)(i) above.

For longitudinal shear the vehicle train in Fig.1(b) shall be applied, see 202(a)(ii) above.

For transverse members the vehicle train in Fig.1(b) or a single 11 ton axle shall be applied as appropriate see 202(b) above.

The vehicle trains shall always be disposed to produce the maximum effect upon the member under consideration.

For carriageway widths less than 18'0" one train of vehicles shall be applied.

For carriageway widths of 18'0" and more, not more than two lanes of vehicles shall be applied, except where crowded traffic conditions call for a greater number, to be decided at the Engineer's discretion. These conditions, which may occur for example when there are traffic lights at one end of the bridge and a bus stop at the other, will have most effect on long span transverse girders.

For the purpose of assessment, the wheel spacing of all vehicles shall be as shown in Fig.2. Where two lanes of vehicles are considered, the distance between lanes shall be taken as 3'0" and the distance between the nearside wheel and the footway or face of parapet where there is no footway, shall be not less than

1'9"; where there is a verge this distance shall be not less than 9".

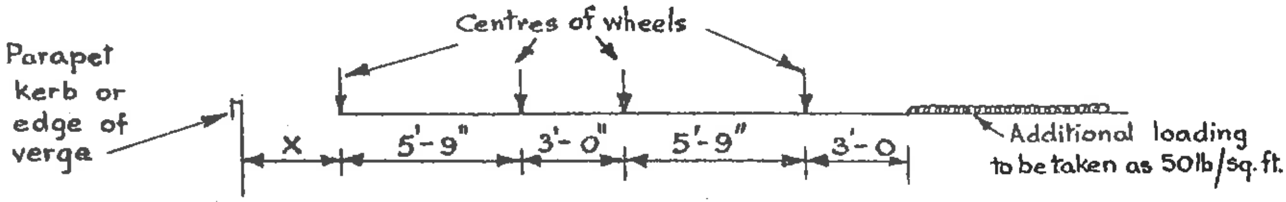


FIG.2. TRACK OF 2 LANES OF EITHER VEHICLE.

X = 1'-9" From bridge parapet where there is no pavement or verge.
X = 1'-9" " kerb where there is a pavement.
X = 9" " edge of verge where there is no kerb.

Note:- Most commercial vehicles have track centres of approx. 5'-9" and an overall track of 7'-9".

(b) Rules for Bridges of Excess Length and Width

Where the length of the bridge exceeds the length occupied by the vehicle trains, a uniformly distributed load of 50 lb/sq ft shall be applied to the remainder of the deck unoccupied by the vehicle train.

Where the carriageway width exceeds 17'6" + X(see Fig.2) a uniformly distributed load of 50 lb/sq ft shall be applied to the excess width when only two trains of vehicles are considered. This additional load need only be considered for cross girders and the total load on an abutment.

(c) Footway Loading

Unless crowd loading is expected, footway loading may be neglected.

(d) Impact

One axle in one vehicle train only shall be subjected to an increase of 25% for impact at the position where it will have the greatest effect on the member being considered.

(e) Contact Areas of Wheels

It may be assumed that all vehicles will have pneumatic tyres. The approximate contact area for each wheel may be obtained by allowing 33 sq in/ton carried by the wheel. The contact area may be assumed as rectangular and equal to $1.4b^2$ where b in inches is the length of the side transverse to the longitudinal axis of the vehicle.

303 Proportion of Load carried by Member

(a) Effective Span

The effective span of a beam shall be taken as the distance between points of support defined as below.

(i) Where a beam is supported by other beams, the web of the supporting beam.

(ii) Where the beam is carried on a purpose made bearing, the centre line of the bearing.

(iii) Where the beam has bearing stiffeners, the point underneath the bearing stiffener.

(iv) Where there are no bearing stiffeners and the beam rests directly on masonry, concrete, or brick, the reaction shall be assumed to be distributed linearly from a maximum at the front edge of the support to zero at the back of the bearing area. The length of the bearing area may be taken as the depth of the beam where the support is of soft brick, reducing to about $\frac{1}{4}$ depth of the beam where the support is of hard material such as concrete or granite. Where the length of support is less than is given by this assumption the assumed bearing area shall be reduced accordingly.

The effective span should be taken as the distance between the centroids of the bearing pressure diagrams.

(b) Dispersion of Loads through Fill

Where fill is well compacted material, dispersion may be taken as 45° from the edge of the wheel contact area. Typical depths to which the dispersion may be taken are:-

(i) Buckle plates:- the highest part of the plate.

(ii) Jack Arches:- the level of the mid depth of the arch ring at the crown.

(iii) Troughing:- the top of the highest flange of the trough.

(iv) Reinforced concrete slabs:- the level of the neutral axis.

(vi) Where adjacent wheel loads dispersed at 45° overlap, the group of wheels or axles may be treated as a whole and the load dispersed at 45° from the centres of the outside wheels or axles of the group.

303 (c) Distribution of Bending Moments by Structural Interaction

(i) The proportion of bending moment carried by longitudinal or transverse members may be determined by the methods given in Part II Section 3, provided the deck between members consists of:-

Reinforced concrete slabs

Jack arches

Concrete in-fill

Buckle plates or cast iron floor plates supported on the bottom flanges of the members and carrying well compacted in-fill.

(ii) Where the skew angle exceeds 35° the proportion factors given in Graphs 1 to 4 shall be multiplied by 1.15.

(iii) The methods given in Part II Section 3, shall not be used:-

For longitudinal girders supporting cross girders.

For any member which does not comply with (i), e.g. a member with simply supported deck plates or slabs resting on the top flange.

For any member spanning between abutments where the direction of the carriageway is at an angle θ greater than 10° and less than 80° to the principal axes of the bridge. (See fig. 3).

In these cases the vehicular loading, including impact, shall be distributed by simple statics between the members to give the maximum proportion of moment on the member under consideration.

303

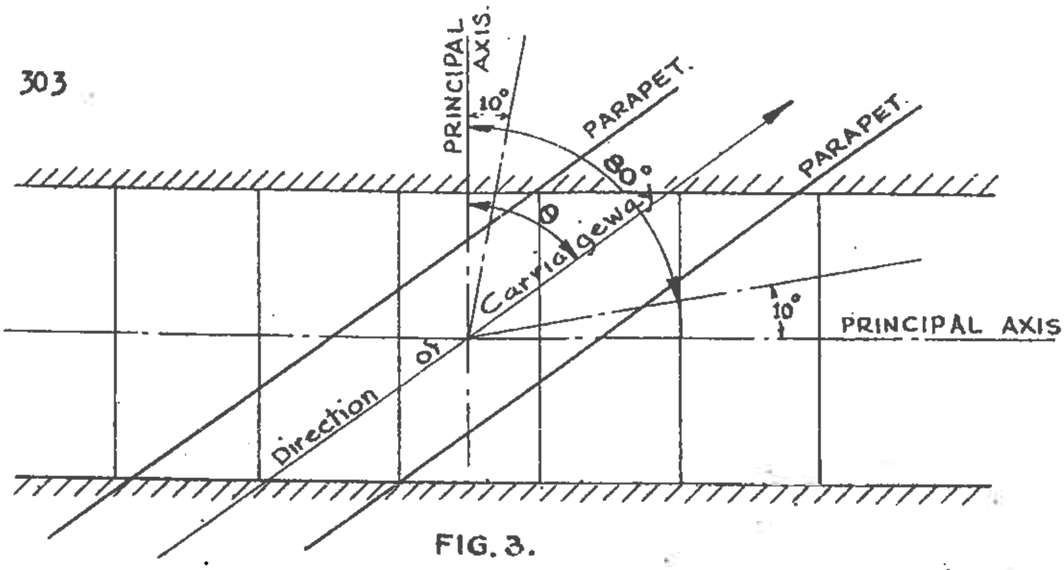


FIG. 3.

(d) <u>Distribution of Shear by Structural Interaction</u>

(i) Provided that the deck complies with sub clause (c)(i) the shear S_L on a longitudinal member 7' or more in length due to trains of loading may be determined from the following formula:-

$$S_L = K(S_V - A) + .625A$$

where S_L = shear on a longitudinal member in tons

K = appropriate proportion factor from graphs 1 to 4

S_V = gross shear of one vehicle train, without impact, in tons.

A = weight of heaviest axle in the train in tons.

Where the span of the longitudinal member is less than 7' the shear shall be calculated assuming statical distribution with impact on one axle.

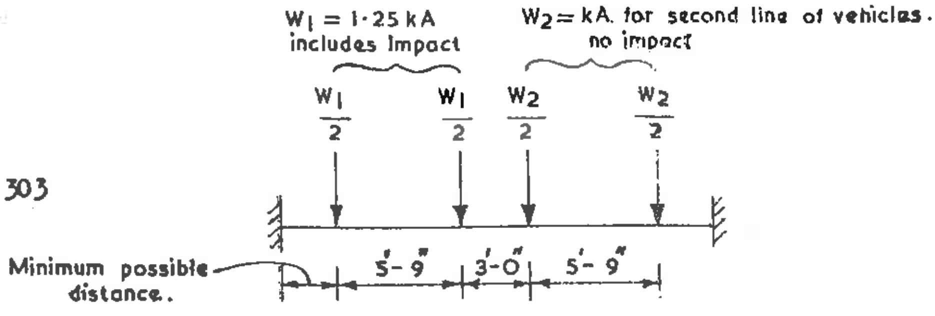
(ii) The shear in a transverse member shall be determined by the application of the appropriate number of lanes of traffic. The effects of one lane may be taken as 1.25 K A and of two lanes as 1.25 K A and K A disposed on the member as shown in fig.4.

A = weight of heaviest axle in the train in tons

K = appropriate proportion factor from graphs 6 and 7

(iii) Graph No.12 gives the gross longitudinal shear for the vehicle train given in fig.1(b) in terms of an axle of unit weight, without impact.

303



SHEAR ON TRANSVERSE MEMBER.
FIG.4.

Where the deck does not comply with sub clause (c)(i) above the shear on both transverse and longitudinal members shall be determined statically with no allowance for distribution.

(e) Troughing.

The load should be dispersed as provided for in sub-clause (b). Provided the troughs are adequately connected, the load may be assumed to be carried by a width of troughing extending equally on either side from a vertical line through the centre of the load for a distance equal to twice the width of the dispersion area. The distribution of load between these troughs shall be taken as linear, being zero at the outer trough and a maximum at the trough under load (Case A in fig.5).

Where the actual troughing does not extend for the distance assumed or where there is a joint of inadequate strength, the amount of load carried by each trough shall be assessed from the ordinates of a distribution diagram as shown in Case B, fig.5. If the edge of the outside trough is stiffened or otherwise supported due consideration may be given to this.

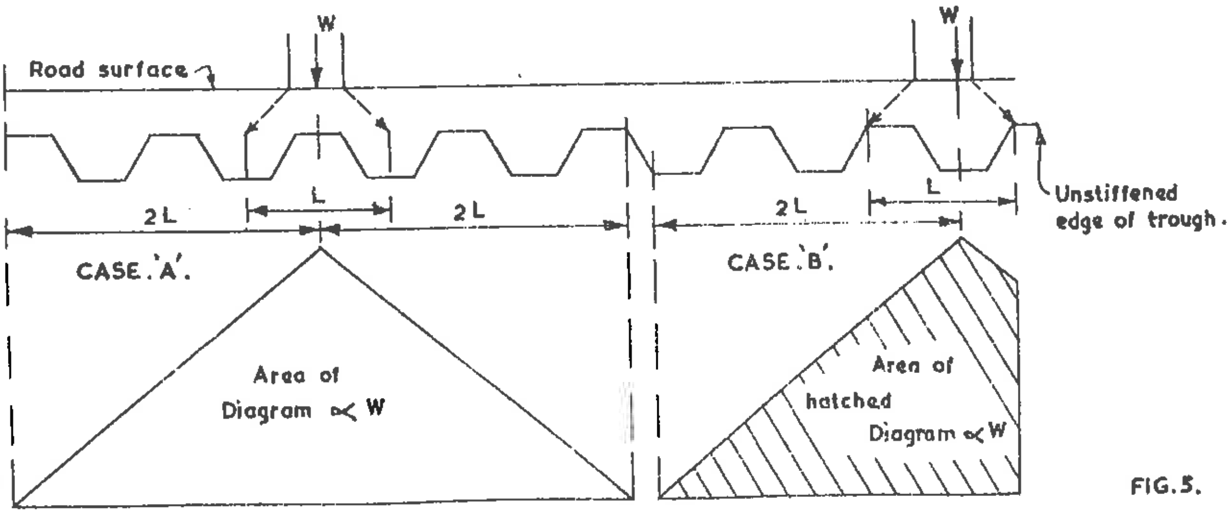


FIG.5.

304 Permissible Stresses

The following basic permissible stresses shall be used as a basis for assessment. They take into account the limited life of the bridge and are subject to the provisos given in Clause 101.

The metallic members of bridges subsequent to 1918 may be assumed to be of good quality steel; prior to that date, they may be of steel, wrought iron or cast iron. The use of steel increased considerably after 1900 but some of it was poor quality and notice should be taken of signs of inferior material e.g. laminations in steel or wrought iron, and blow holes in cast iron.

In cases of particular importance, it may be necessary to have the material tested.

The formula for the permissible stress in cast iron contains a factor for fatigue. The stresses in steel and wrought iron may need to be reduced where a bridge is on a heavily trafficked route and contains small members which carry little dead load and are highly stressed by a single wheel or axle.

(a) Mild Steel

The stresses shall be as in B.S.153, Part 3B, Table 1, Case II.

(b) Wrought Iron

The stresses shall be as for mild steel multiplied by the ratio of their respective yield stresses. A yield stress of 10.75 ton/sq in is generally attained, giving a basic permissible stress in bending of $\frac{10.75}{16} \times 12.5 = 8.4$ ton/ sq in

Where there is reason to believe that the yield stress may be lower tests shall be made, or if this is not possible, a lower value shall be estimated and the allowable working stress decreased pro rata.

(c) Cast Iron

Up to 3 tons/sq in tension. The actual permissible tensile stress shall be based on the ratio of dead to live load, and calculated from the equation $5f_L + 2.2\, f_D = 8$, but shall in no case exceed 3 tons/sq in. Permissible stress in compression 10 ton sq in. The shear stress is not critical in a cast iron beam because of the thickness of the web in relation to the other dimensions.

(d) Reinforced Concrete

Permissible stresses for bridges built prior to 1922 are

Bending Compression	Shear	Bond	Steel
lb/ sq in	lb/sq in	lb/sq in	lb/sq in
750	75	100	20,000

Where bridges as defined in clause 301(a) have deteriorated so that assessment is necessary the same stresses should be used, and allowance made for loss of section as appropriate.

305 Calculation of Strength of Members

(a) General

Cast iron members need be checked only in bending, see clause 304(c). All other members, shall be checked for bending and shear and the lower assessment figure taken.

Simply supported longitudinal members not exceeding 60' span and 7' spacing, having a live load capacity in shear equal to or greater than the values given in Table 1, will safely carry two trains of 24 ton vehicles.

Where the available shear is less than that given in the table, actual limiting axle weights shall be calculated according to the formula given in clause 303(d)(ii).

Table 1

Span ft	Shear Value Tons
15 and below	10
20	11
25	11.5
30	12.5
35	14
40	15.5
50	17.5
60	20.5

(i) Flange and Web Splices

It will be frequently found that in old bridges one or other of these may govern the strength. It is important therefore that both are always considered

(ii) Rivets

In cases where alternating loads occur on rivets attention is drawn to Clause 24e of B.S.153, Part 3B.

(iii) Corrosion

Where inspection is possible, an allowance shall be made for any reduction in section. Otherwise, appropriate reductions in permissible stress shall be estimated.

(iv) Cracks and other faults

Note should be taken of cracks, flaws and any other faults in the superstructure or substructure and due allowance made for the adverse effect of these in assessment.

(b) Methods of Calculation

(i) Mild Steel and Wrought Iron:-

Calculations for deck and column members shall be based on the current B.S. 153 for Steel Girder Bridges. Where bridge details do not comply with B.S. 153 the variations shall be taken into consideration and due allowance made in assessment.

(ii) Cast Iron

(1) Girders

The section modulus of the girder may be increased for live load by the factor $\frac{D}{d}$ where D is the overall depth of the deck less 3" for surfacing material and d is the depth of the bare girder at midspan provided:-

The girders are known to be firmly embedded in well consolidated filling material (except where such material is pure sand or pure clay).

There are no services in the carriageway which would decrease the support rendered by the fill e.g. stoneware pipes or large diameter water or gas mains.

Should openings be made in the carriageway after an assessment which used the $\frac{D}{d}$ factor, the opening must be back filled with concrete, or the assessment reconsidered.

(2) Columns

Cast Iron columns shall be calculated by the Gordon-Rankine equation:-

$$\text{Safe load } p = 5\left(\frac{f_c A}{1 + Fa\,\frac{L^2}{K^2}}\right) \text{ tons}$$

where f_c = compressive yield stress, 36 ton/sq in

A = cross sectional area, sq in

L = length in

K = least radius of gyration, in

F = end fixity factor

a = material factor, $\frac{1}{1600}$

Values of F:-	
Both ends pin jointed:-	1
One end fixed, one end pin jointed	½
Both ends rigidly fixed	¼
One end fixed, one end entirely free	4

(iii) Reinforced Concrete

Calculations for routine assessments shall be based on C.P. 114: subject to the provisos of Clause 304(d) of this Assessment Code.

(iv) Brick and Masonry Arch Rings

Guidance on the assessment of brick and masonry arch rings is in course of preparation and will be issued when complete.

In general such arches have a large reserve of strength and will normally be adequate for H.A. loading provided they have been freely used by the full range of Construction and Use Vehicles and are in good condition. When the Engineer is in doubt advice should be sought from the Bridges Engineering Division of the Ministry of Transport.

SECTION 4 RELATION OF ASSESSMENT TO WEIGHT RESTRICTION AND ERECTION OF SIGNS

401 Difference in Capacity

In some cases e.g. widened bridges, there may be some difference between the capacity of one part of the bridge and the remainder. The assessment shall be made on the basis of the weaker part, and a note on the strength of the stronger part should be given if strengthening measures are to considered later.

402 Public Service Vehicles

Because of its different dimensions and weight distribution, a bus may be heavier than an assessment vehicle and have no greater effect in bending. Table II gives the equivalent vehicle weight assessment at various spans, for typical buses available at the date of issue of this Code. Where the effect of the bus is no greater than the vehicle weight assessment, then the bus may be excluded from the vehicle weight restriction order.

Where a bridge is restricted by an axle weight, a separate investigation may be necessary using the actual axle weights and spacings given in Table II. It may be assumed that the distance from the front or rear axles of the bus, and the nearest axle of the adjacent vehicle is 10'0".

Table II

Gross Weight and make of P.S.V. (tons)	Axle Spacing	Weight Distribution (tons)	Equivalent Weight of Assessment Vehicle (tons)											
			Span											
			5'	10'	15'	20'	25'	30'	35'	40'	45'	50'	55'	60'-100'
6.5 Bedford	13'8"	2.6, 3.9	6	6	6	6	5	5	5	5	5	5	5	5
8.75 Dennis	16'2"	3.75, 5.0	8	8	8	7	7	6	6	6	6	6	6	6
9.95 A.E.C.	18.7"	4.35, 5.6	9	9	9	8	7	7	6	6	6	6	6	6
10.2 Bedford	15'4"	5.1, 5.1	8	8	8	7	7	7	7	6	6	6	6	6
10.5 Bedford	4'0" 15'3"	3.25, 3.25, 4.0	6	7	7	7	7	7	7	7	6	6	6	6
10.9 Leyland	18'6"	3.45, 7.45	12	11	11	11	10	9	8	8	7	7	7	7
13.05 Daimler	18'6"	4.05, 9.0	13	13	13	13	11	11	10	9	9	8	8	8
13.2 A.E.C.	18'7"	4.6, 8.6	13	13	13	12	11	11	10	10	9	9	9	8
13.35 Daimler	18'6"	5.55, 7.8	12	12	12	12	11	11	11	10	10	9	9	9
13.5 Daimler	16'3"	4.75, 8.75	13	13	13	13	11	11	10	10	10	9	9	8
13.5 Leyland	16'3"	4.55, 8.95	13	13	13	12	11	10	10	10	9	9	9	8
14.0 Daimler	18'6"	5.0, 9.0	13	13	13	12	11	11	10	10	9	9	9	8
14.0 Leyland	16'3"	4.0, 10.0	14	14	14	14	13	12	11	11	10	10	9	9

402 Apparent inconsistencies between vehicles e.g. the 10.2 ton and 10.5 ton Bedfords, are due to the differences in number of axles and the distribution between them.

403 Levels of Assessment for Longitudinal Members in Bending

As described in Clause 202(a)(i) a longitudinal member which is capable of carrying in bending the 2 axle 14 ton vehicle will be able to carry all permitted vehicles having a gross weight not exceeding 20 tons, and any member capable of carrying the 4 axle 24 ton vehicle will be able to carry all other vehicles complying with the Construction and Use Regulations current at the date of issue of this Code. Assessments should therefore be in one ton intervals up to 13 tons, then at 20 tons, 22 tons, and 24 tons. (NOTE. The maximum level of assessment at which a statutory weight restriction should be imposed has not yet been prescribed.)

404 Levels of Assessment for Transverse Members

As described in Clause 202(b) any transverse member which can carry the two 9 ton axles with the associated axles derived from Fig.1(b), will be capable of carrying the single 11 ton axle with its associated axles permitted by the Construction and Use Regulation current at the date of issue of this Code, subject to conformity with the requirements of Clause 303(c)(i). For members satisfying these requirements axle restriction should therefore not be applied above 8 tons, and up to this value should be in 1 ton intervals. The restrictions for both bending and shear will be in terms of the axle load the member can safely carry.

405 Levels of Assessment for Longitudinal Members in Shear

An axle load restriction will be applied and will be in one ton intervals up to eight tons as for transverse members, see Clause 202(a)(ii).

406 Comparison of Capacities

The capacity of a bridge deck may be limited in one or more of the following conditions:-

(a) Longitudinal bending

(b) Longitudinal shear

(c) Transverse bending or shear

Where (a) is the only criterion, the restriction will be in terms of vehicle weight.

406 Where (b) or (c) is the only criterion, the restriction will be in terms of axle weight based on the train of vehicles given in Fig.1(b) since this is less restrictive to traffic as a whole than a restriction based on vehicle weight.

Where the restriction is a choice between

(i) (a) or (b) above

(ii) (a) or (c) "

the corresponding axle weight of the limiting vehicle shall be calculated as 0.7 of the limiting vehicle weight, or 9 tons whichever is the lesser, and compared with the limiting axle weight. Restrictions shall then be applied on the basis of this comparison as follows:-

(iii) Where the corresponding axle weight is less than or equal to the limiting axle weight, a vehicle weight restriction only need be applied.

(iv) Where the corresponding axle weight is greater than the limiting axle weight a vehicle weight restriction and an axle weight restriction shall be applied except

(v) Where the limiting axle weight is less than 0.3 the limiting vehicle weight, when only an axle weight restriction need be applied. (In the trains given in fig. 1 no vehicle will weigh more than 3 times its heaviest axle. In practice, the weight distribution may be more even and a four axle vehicle might weigh 3.3 times the weight of its heaviest axle).

407 Restriction Signs

The sign restriction(s) finally selected shall comply with fig. 3.36 and 3.38 in the Traffic Signs Manual (H.M.S.O. 1965). A supplementary sign fig. 3.37 "Only one vehicle on bridge" is also permissible, but its use should normally be restricted to cases where traffic control is exercised, e.g. during a bridge reconstruction. The loading for this particular case has not been considered in Section II.

SECTION I SCOPE

101 The method applies to simply supported decks containing longitudinal or transverse members which satisfy the conditions of Clause 303(c), in Part I. It does not apply to the parapet girder supporting the transverse cross girders for which separate calculations must be made.

It determines the value of the beam in bending only, but where necessary the factors may be used when checking for shear. See Part I Clause 303(d).

SECTION 2 THEORY

201 General Principle

(a) Longitudinal Members

When a bridge deck with longitudinal members is loaded with one or two lanes of vehicles as described in Part I Clauses 202(a) and 302(a) the vehicles can be disposed to produce a maximum effect on one longitudinal member. The moment induced in the member by either one or two lanes, can be expressed as a product of the moment caused by one train of vehicles (MV) and a factor K termed the proportion factor. If the live load moment of resistance of the member equals MR then MR must $\geqslant$ K MV or more conveniently $\frac{MR}{K} \geqslant MV$ for the member to be safe.

(b) Transverse Members

In the case of transverse members, the principle is similar but here the K factor gives the proportion of transverse load carried by a cross girder under one or two vehicle trains of the type shown in Fig.1b of Part I, in terms of the heaviest axle.

202 Derivation of Proportion Factors

The proportion factors given in the Code are based on the theoretical approach by Jaeger & Hendry (described in their Book; the Analysis of Grid Frameworks and Related Structures) and the test results and recommendations given by Thomas & Short of the Building Research Station published in the I.C.E. Journal for March 1952. Referring to Jaeger & Hendry's book (see page 19) it was found that using a value for $\frac{EI_T}{EI}$ of .0305 the resulting value combined with $B = \infty$ gives exact correspondence with the distribution in the model jack arch bridge described in Thomas & Short's paper and fair correspondence for two bridges which had been tested in 1943.

Proportion factors for internal and external girders were therefore calculated by Jaeger and Hendry's method for various girder spacings and spans using this constant value for $\frac{EI_T}{EI}$ for spans of up to 30' longitudinally and 35' transversely.

Because of the absence of experimental data the values for proportion factors at these spans were maintained for all greater spans. This will err in the direction of safety.

The different approach of Thomas & Short was then used to calculate similar sets of proportion factors which were plotted in conjunction with the previous sets.

Finally envelopes were drawn embracing the two sets of values obtained, and the factors produced are to be found in graphs 1-4 and 6-9.

203 Impact

(a) Longitudinal Members

The proportion factors have been calculated with no allowance for impact. The graph for longitudinal bending moment due to one lane of vehicles includes an allowance for 25% impact on one axle.

As this graph is also used for assessing bridges carrying two lanes, allowance is therefore made for the effects of impact in each lane, but the error will be small, as the effect of the second lane is much less than that of the first lane.

(b) Transverse Members

The transverse bending moment due to one or two axles includes no allowance for impact as this is built into the proportion factors.

SECTION 3 METHOD OF USE OF DISTRIBUTION CURVES

301 Longitudinal Members

The internal members shall always be investigated, but the edge members need only be examined when the position of the near side wheels in the vehicle train as determined by Fig. 2 is such that they have no other member between them and the edge member.

The steps to establish the assessment of a longitudinal member are as follows:-

(a) Bending

(i) Determine the moment of resistance, of the member available for live load and if the span is greater than 70' deduct the moment due to the distributed loading of 50 lb./sq. ft.

(ii) Find the value of K from graphs 1 to 4.

(iii) Divide (i) by (ii).

(iv) Transpose the value obtained in (iii) to the relevant span in graph 5 to obtain the assessment. Where the bridge is skew the relevant span shall be taken as the skew span.

(b) Shear (If available live load shear is less than the appropriate value in Table 1)

(i) Determine the shear available for live load and if the span is greater than 70' deduct the shear due to the distributed loading of 50 lb./sq. ft. acting at the "far end" of the span.

(ii) Subtract 0.625 times the maximum axle load from this available shear.

(iii) Find the appropriate value of K and divide (ii) by this.

(iv) Add the maximum axle load to (iii).

(v) Divide by the appropriate shear from graph 12 to obtain the axle weight assessment.

302 Transverse Members

Both internal and external members shall be investigated.

The steps to establish the assessment of a transverse member are as follows:-

302 (a) Bending

(i) Determine the moment of resistance of the member available for live load. Where the carriageway has more than two lanes, deduct the 50 lb. per sq. ft. U.D. load (see Clause 302(b) Part 1.)

(ii) Find the relevant value of K from graphs 6 to 9.

(iii) Divide (i) by (ii).

(iv) Transpose the value obtained in (iii) to the relevant span in graphs 10 or 11 for transverse bending moment to determine the assessment of the tranverse member in terms of axle weight.

(b) Shear

(i) Determine the shear available for live load and deduct the shear due to the 50 lb./sq. ft. loading as in 302(a).

(ii) Find the appropriate value of K from graphs 6 to 9.

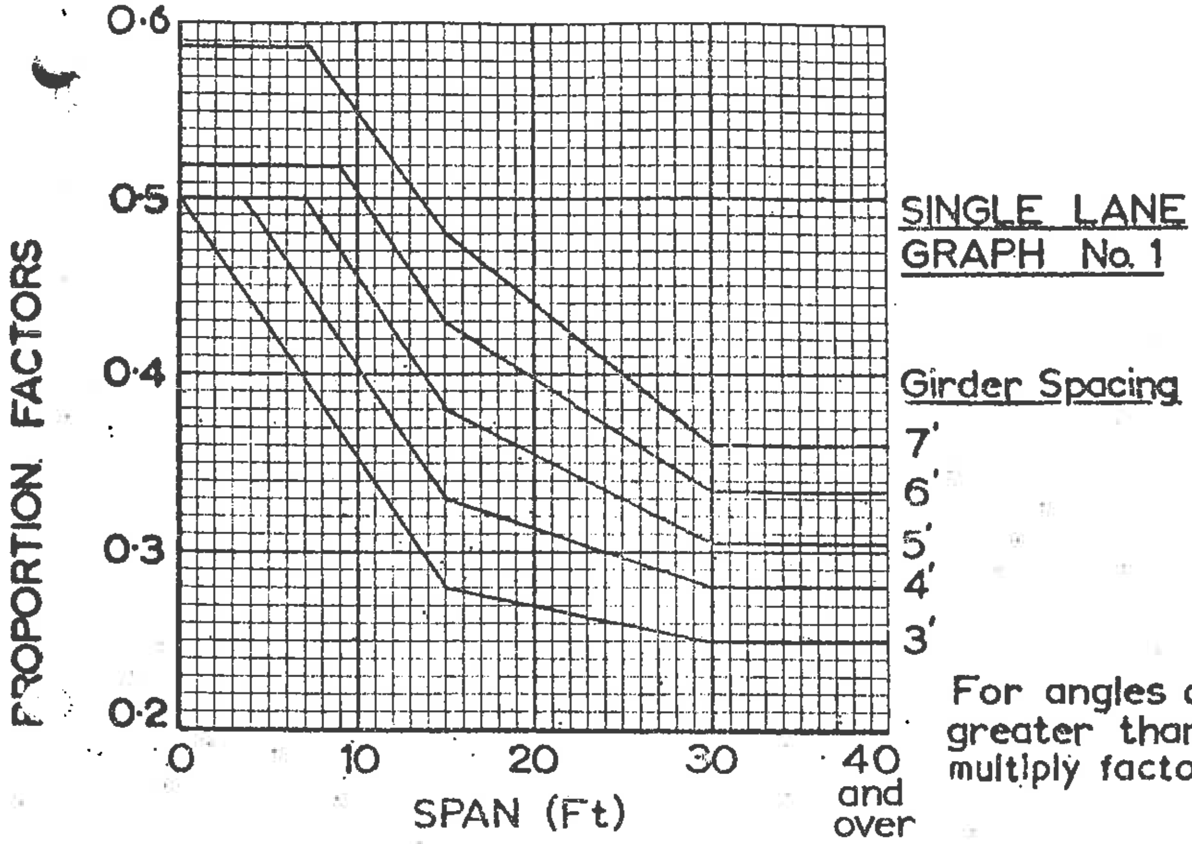
(iii) Determine shear due to trains with unit maximum axle load.

(iv) Divide (i) by (iii) to obtain the axle weight assessment.

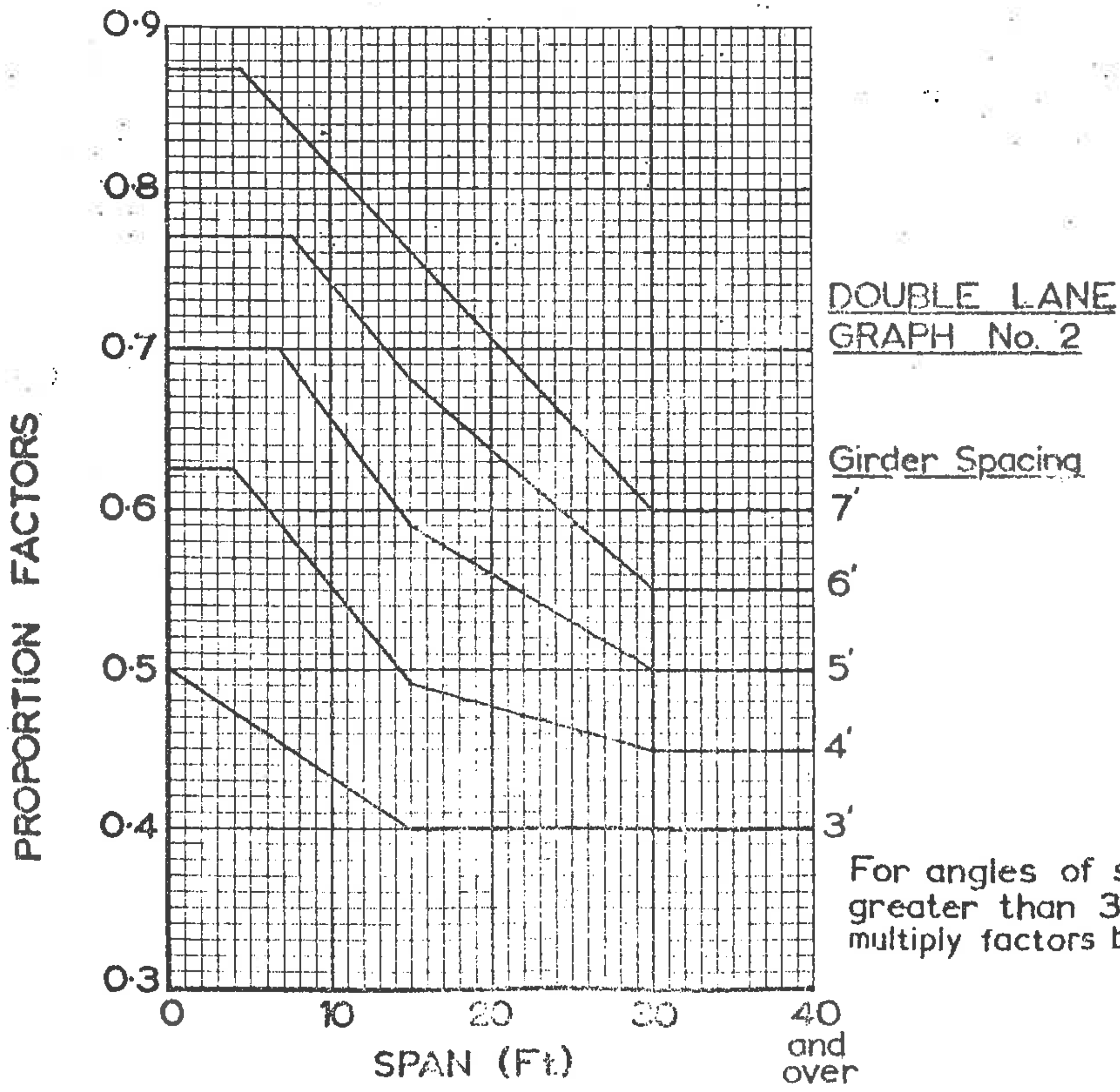
303 Final Assessment

After all members have been assessed the procedure described in Part I, Section 4, shall be followed.

PROPORTION FACTORS FOR INTERNAL LONGITUDINAL GIRDERS.

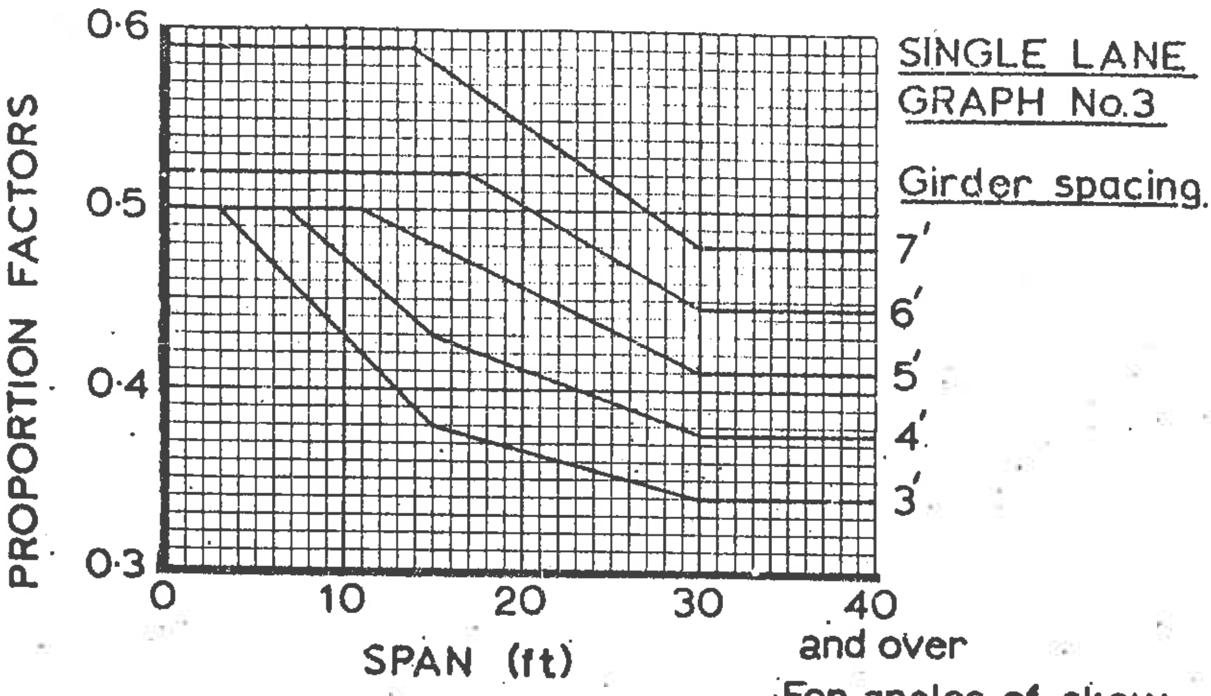


For angles of skew greater than 35° multiply factors by 1·15

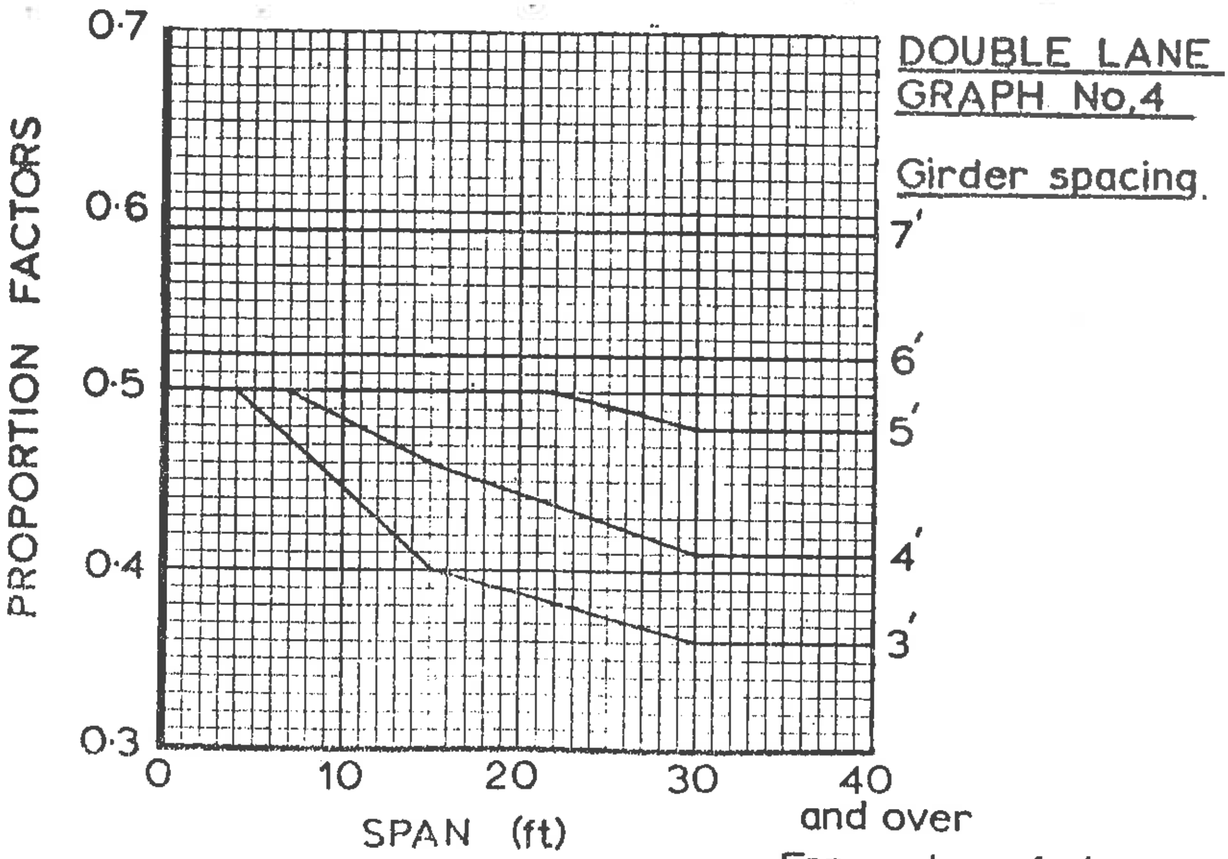


For angles of skew greater than 35° multiply factors by 1·15

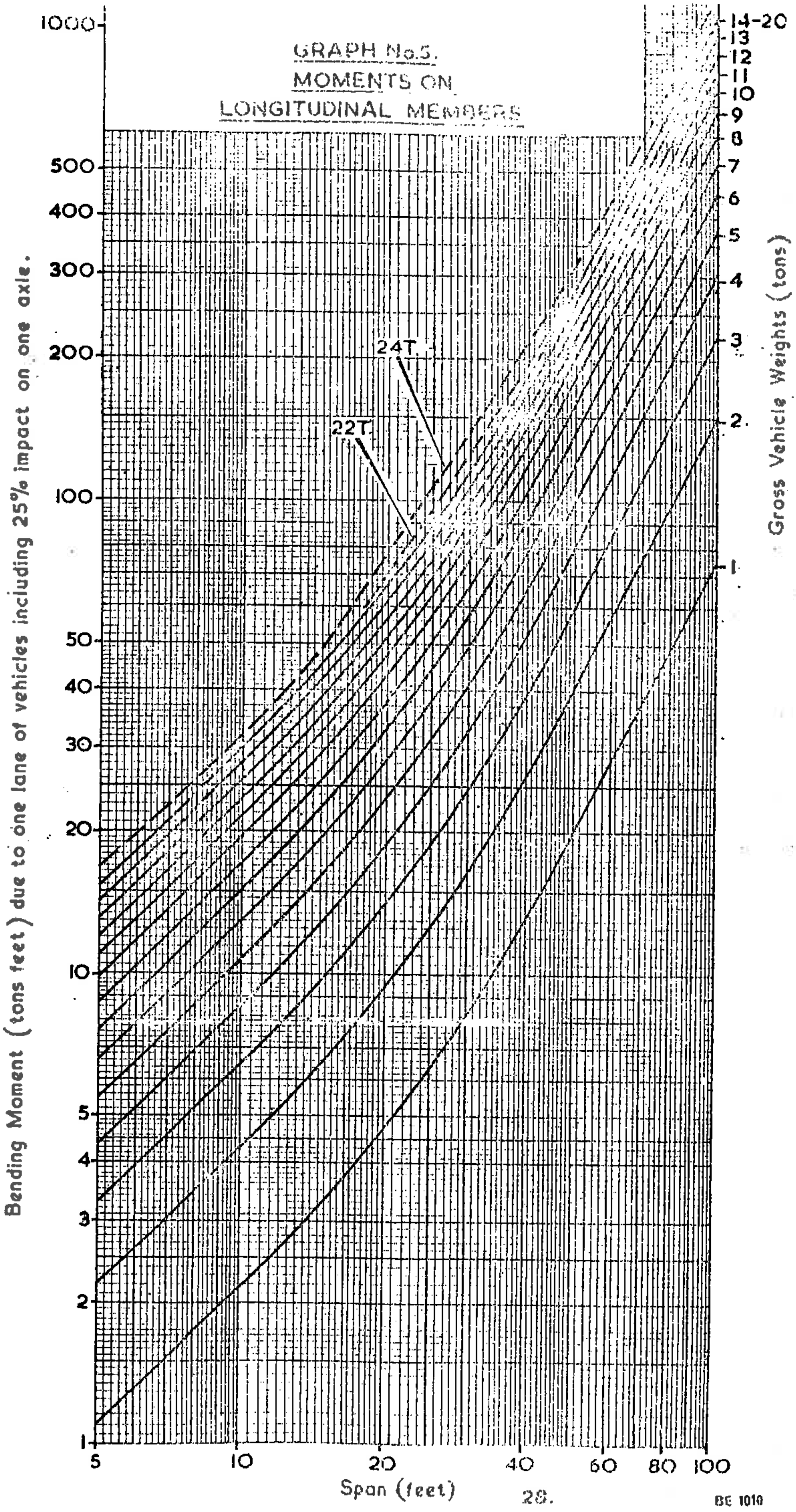
LONGITUDINAL GIRDERS.



For angles of skew greater than 35° multiply factors by 1·15

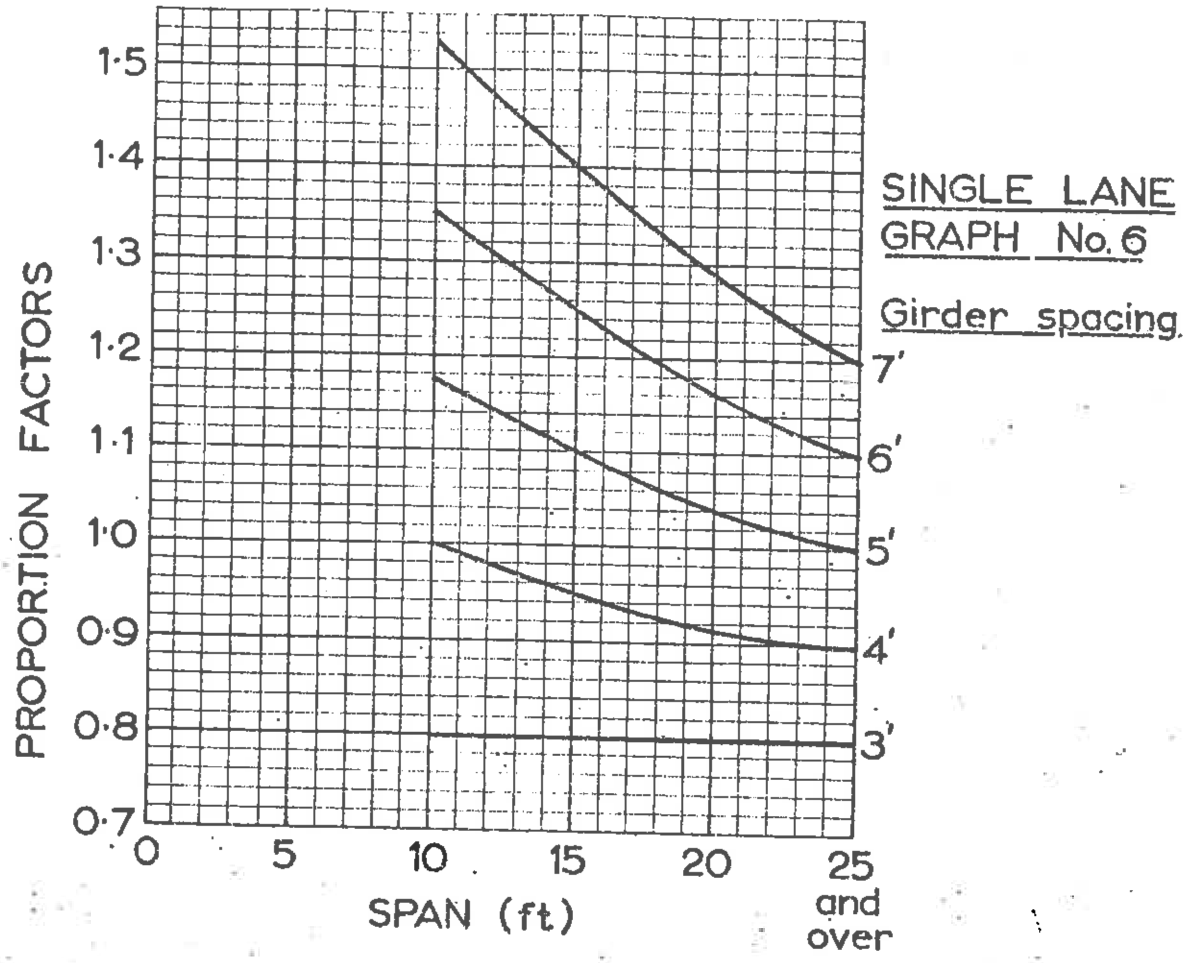


For angles of skew greater than 35° multiply factors by 1·15

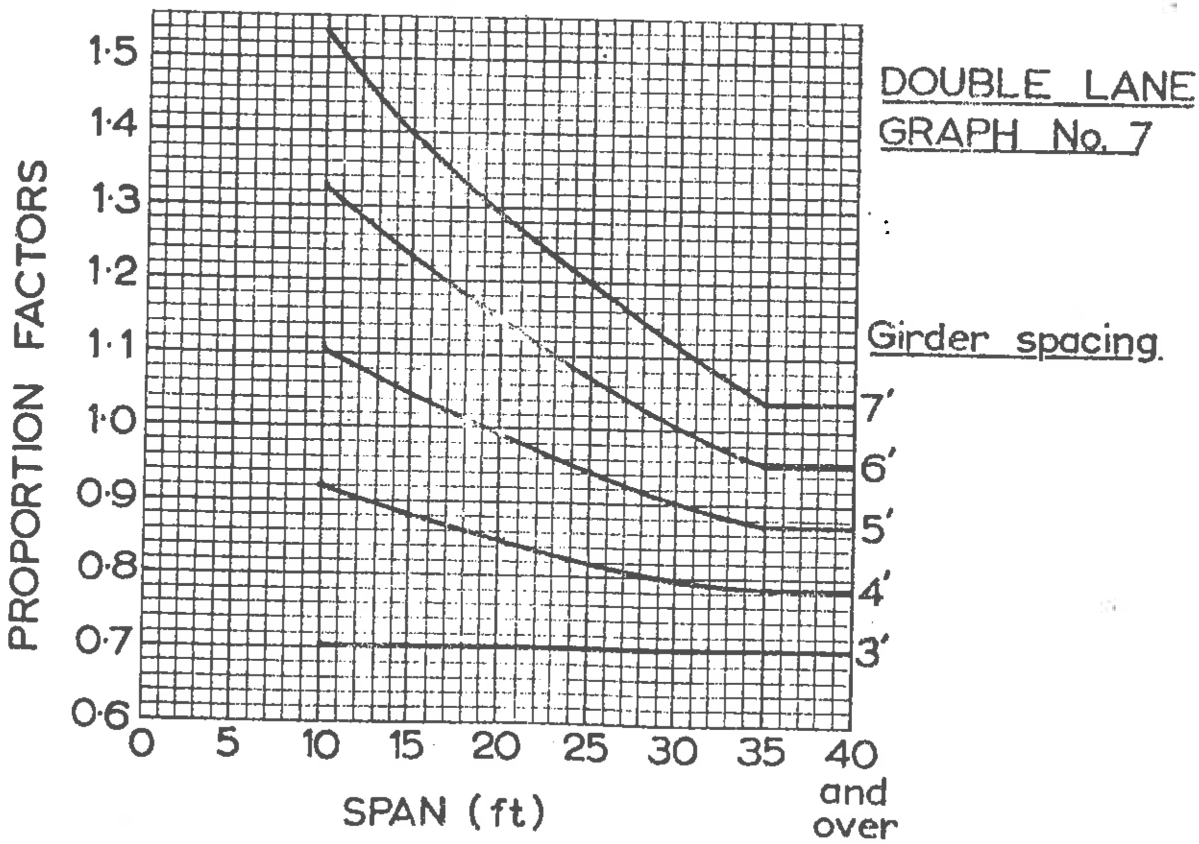

GRAPH No.5.
MOMENTS ON
LONGITUDINAL MEMBERS
24T.
22T.
Bending Moment (tons feet) due to one lane of vehicles including 25% impact on one axle.
1000
500
400
300
200
100
50
40
30
20
10
5
4
3
2
1
Span (feet)
5
10
20
40
60
80
100
Gross Vehicle Weights (tons)
14-20
13
12
11
10
9
8
7
6
5
4
3
2.
1.

BE 1010

PROPORTION FACTORS FOR INTERNAL TRANSVERSE GIRDERS



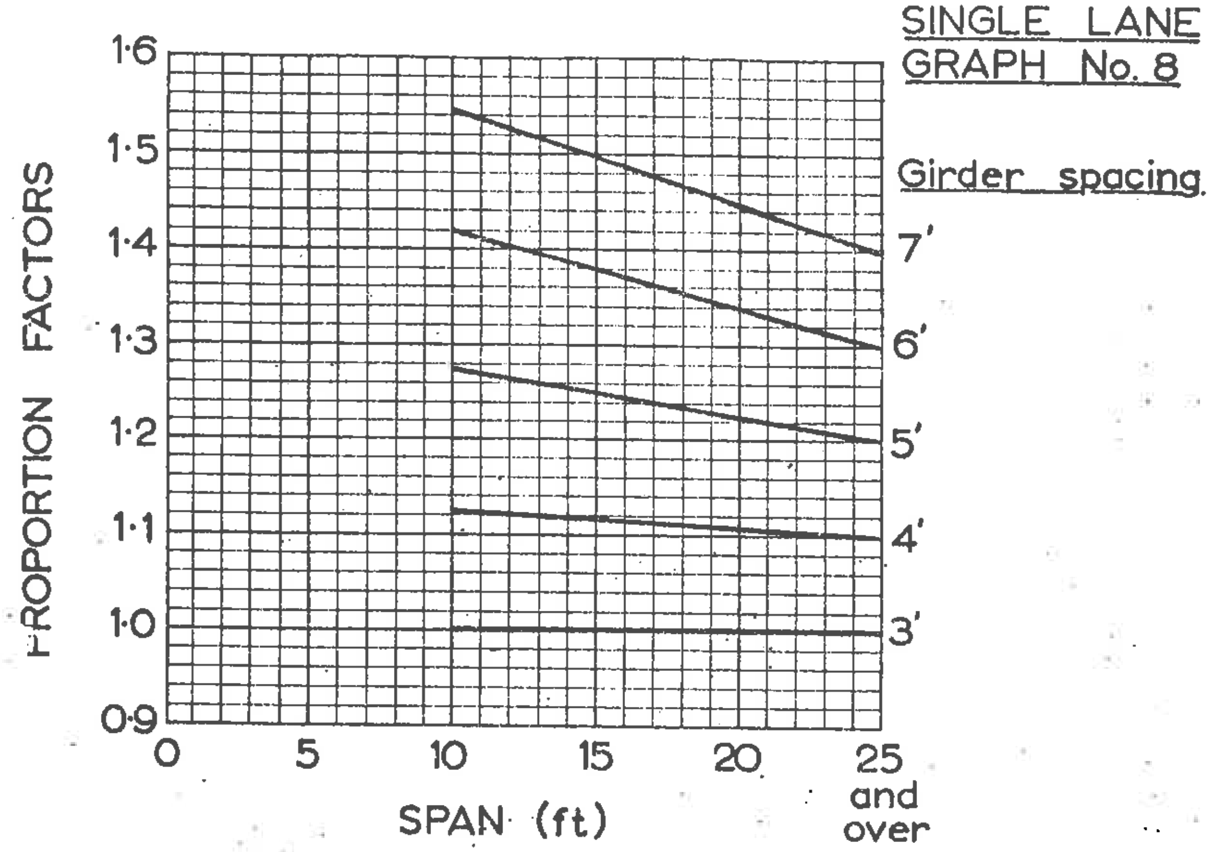
SINGLE LANE GRAPH No. 6



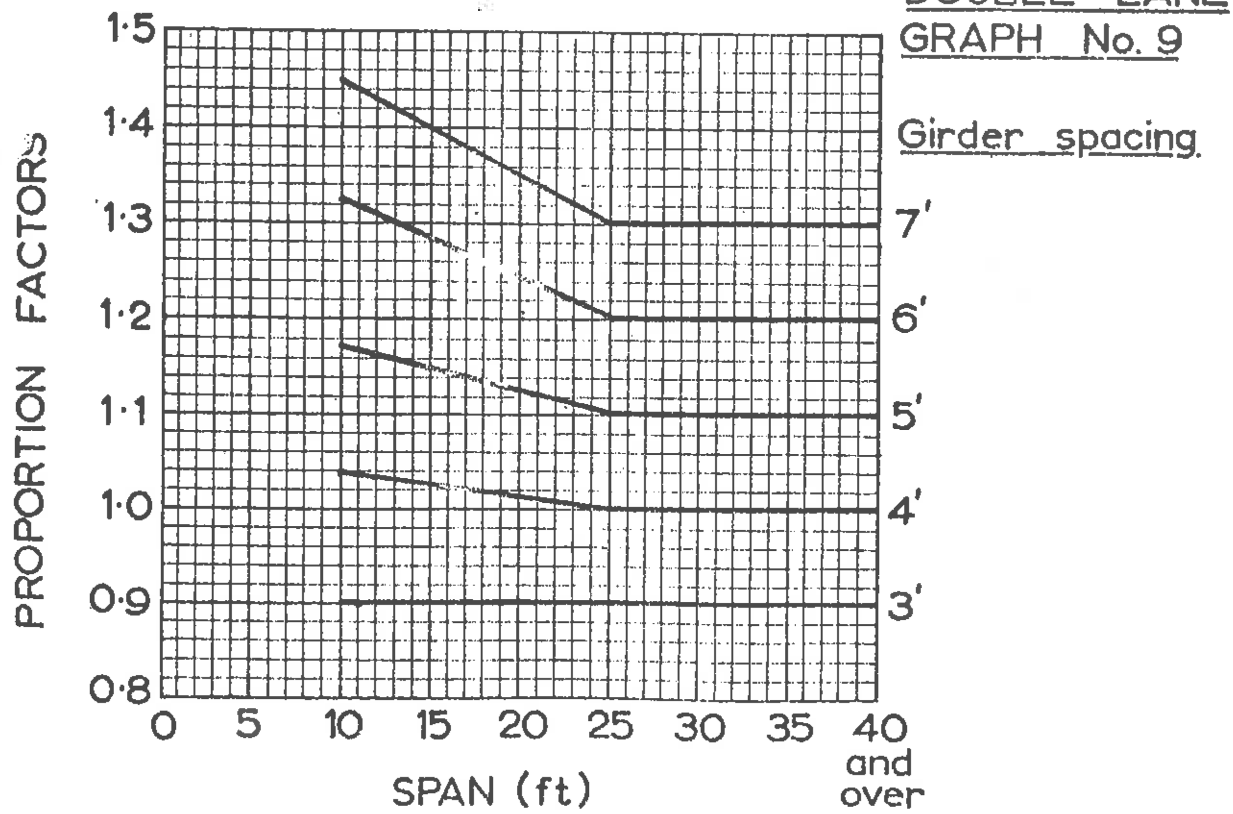
DOUBLE LANE GRAPH No. 7

PROPORTION FACTORS FOR EXTERNAL TRANSVERSE GIRDERS

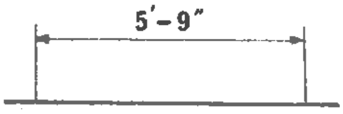
SINGLE LANE
GRAPH No. 8



DOUBLE LANE
GRAPH No. 9



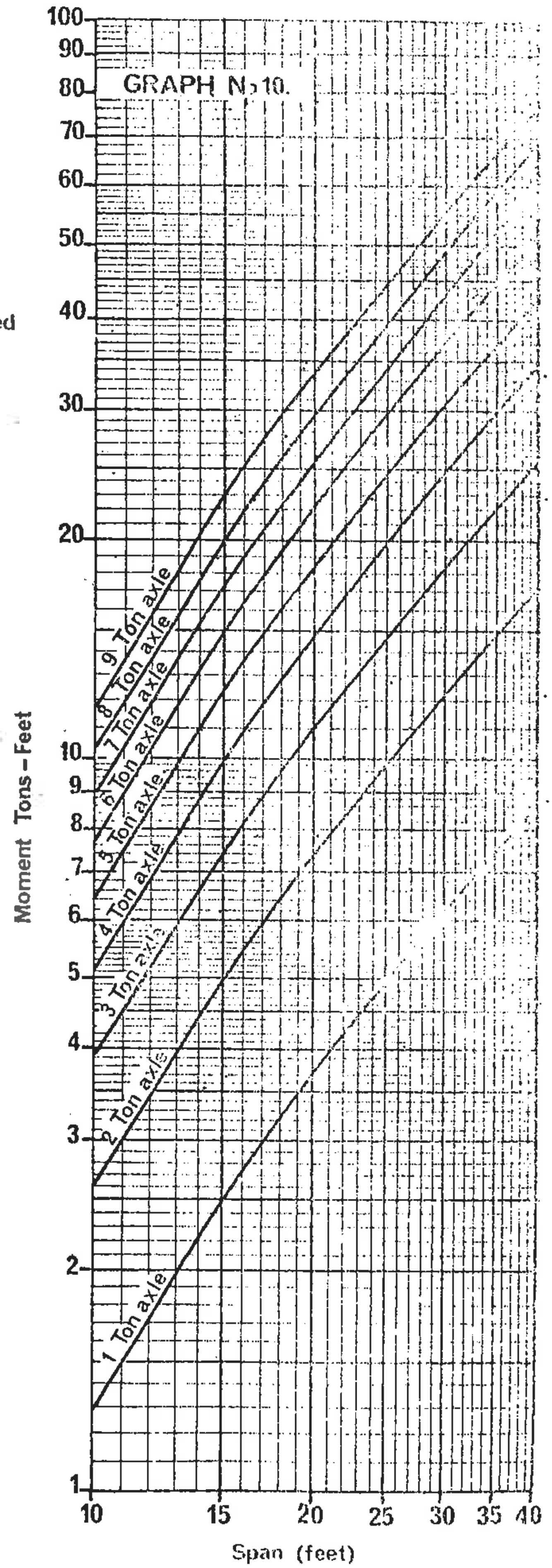
INDIVIDUAL AXLE (SINGLE TRAIN)



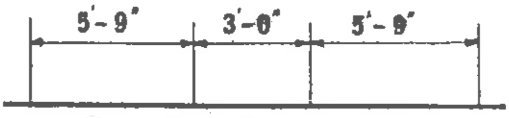
Wheel Centres

Wheel arrangement is assumed to be disposed to give max. moment.

Graphs are drawn for 1 ton increments per axle



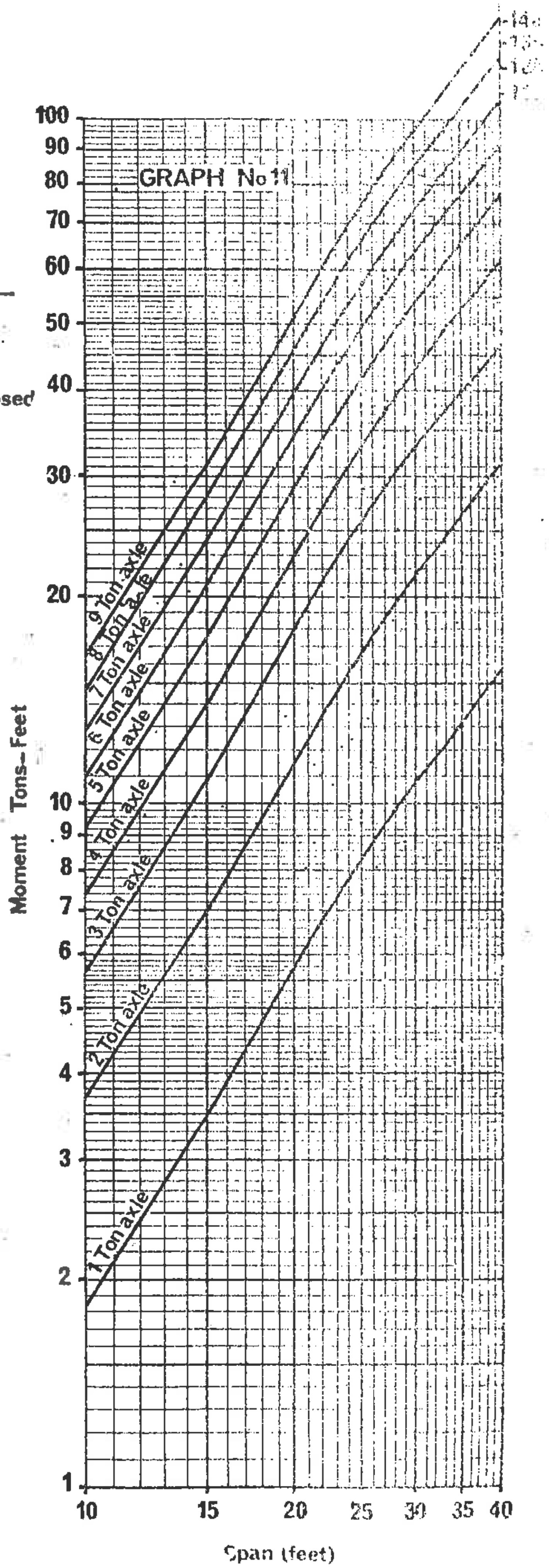
FOR INDIVIDUAL AXLES (DOUBLE TRAIN)



Wheel Centres

Wheel arrangement is assumed to be disposed to give max. moment.

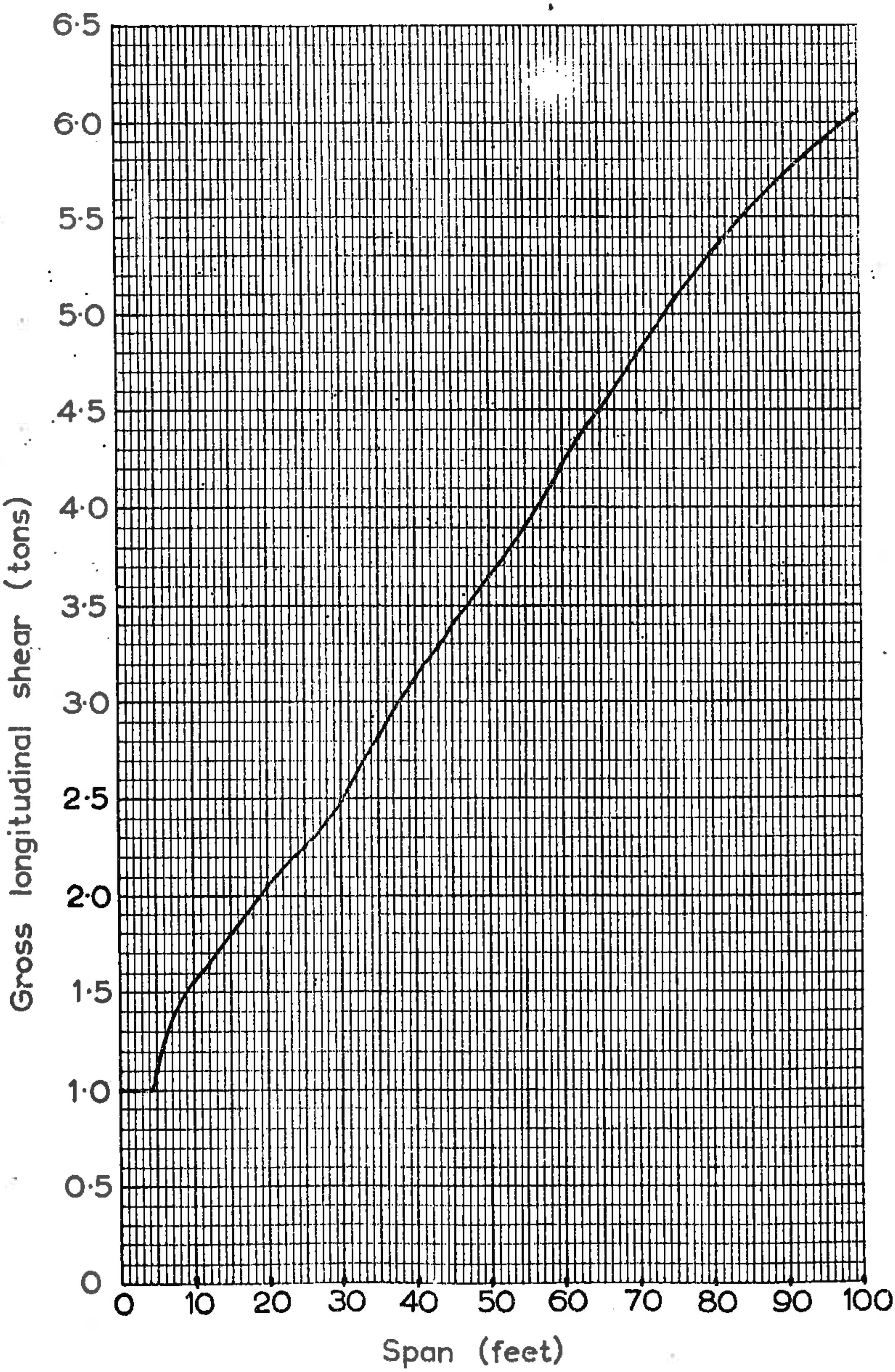
Graphs are drawn for 1 ton increments per axle



GRAPH No. 12

TRAIN OF VEHICLES TYPE 1(b)

GROSS LONGITUDINAL SHEAR FOR ONE TRAIN WITH UNIT MAXIMUM AXLE LOADING



ASSESSMENT OF MASONRY AND BRICK ARCH BRIDGES

1. Scope

The following conditions in Part I:- Basic Principles shall also apply to Part III:-

Clause 101	Scope	In full
Clause 201:	Regulation of Vehicles	In full
Clause 302(a):	Application of Vehicle Trains	The number of trains/ lane width relationship only.
Clause 302(d):	Impact	Allowed for in nomogram
Clause 305(b)(iv):	Brick and Masonry Arch Rings	In full
Clause 401:	Difference in Capacity	In full
Clause 407:	Restriction Signs	In full

2. Method of Assessment

The assessment of arch bridges has been adapted from the method set out in "Military Load Classification by the Reconnaissance and Correlation Methods", M.E.X.E. May 1963. This method is based on the results of past experience, and it has been found to give satisfactory results for the range of vehicles conforming to the Construction and Use Regulations but it's extrapolated use for heavier vehicles, or for spans greater than 60' should be treated with caution and is subject to the consent of the Authority concerned. It is intended to be applied primarily to single span arches and in the case of multiple spans, particular attention is drawn to clause 8(e)(vii).

The assessments are in terms of a maximum allowable axle load, such a rating allowing for twin axles.

A rating of 9 tons therefore means that a bridge is capable of carrying vehicles with tandem 9 ton axles, and since such a bridge is also capable of carrying vehicles with a single 11 ton axle, it follows that arch bridges with a final assessment of 9 tons or more are unrestricted for all vehicles complying with Construction and Use Regulations current at the date of issue of this document, and there will be no axle weight restriction signs above 8 tons.

3. Public Service Vehicles

Public Service vehicles, other than those with tandem axles, may have a permitted axle weight equal to 5/4 the assessed axle weight rounded up to the nearest whole number.

The real strength of a masonry or brick arch bridge is almost impossible to calculate, and recourse has, therefore, been made to an empirical formula based on the bridge dimensions. The bridge is first assumed to be soundly built in good-quality brickwork, with well pointed joints, to be free from cracks, and to have adequate abutments. For such an idealised bridge, a provisional assessment is obtained from a nomogram (Graph No.13). This provisional assessment is then modified by factors which allow for the way in which the actual bridge differs from the ideal.

5. Measurement of Dimensions

The following dimensions must be measured:-

(a) The span .... l ft.

(in the case of skew spans, measure l parallel to the axis of the arch)

(b) The rise of the arch ring at the crown .... r_c ft

(c) The rise of the arch at the quarter points .... r_q ft

(d) The thickness of the arch ring at the crown .... d ft

(e) The average depth of fill between the road surface and the arch ring at the crown, measured .... h ft at the quarter points of the transverse road profile.

Fig. 6 shows where these dimensions are measured.

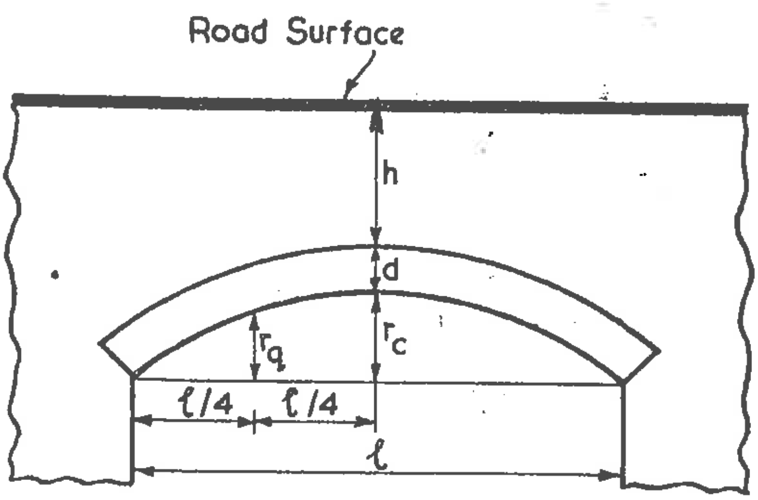


FIG. 6.

6. Examination of the Bridge

Normally only the external fabric needs to be examined. Probing into the construction will only be necessary on important routes where the strength of the bridge is in doubt.

(a) The Arch Ring

(i) Nature and condition of the brickwork or masonry.

(ii) Thickness of the joints.

(iii) Condition of the mortar.

(iv) Deformation of the arch ring from its original shape.

(v) Presence of cracks - their width, length, number and position.

(b) Parapet and Spandrel Walls

(i) Sagging of the parapet.

(ii) Cracks in the parapet or spandrel walls.

(iii) Outward movement of the parapet or spandrel relative to the face of the arch ring.

(c) Abutments

Damage to the abutments and wing walls by cracking, settlement or movement.

(d) Sources of Error

(i) The thickness of the arch ring under the parapet can be measured, but it does not follow that this thickness obtains under the roadway.

(ii) Some old bridges have been strengthened by removing the fill and replacing it with concrete.

(iii) Services which are laid over or through the arch ring may affect the strength. The position and size of these should be determined and where they are large or numerous allowance should be made when assessing the condition factor (see Clause 8(e)). Where there is any doubt as to any of the above conditions an investigation on site, including the digging of trial holes, may have to be made.

7. Provisional Assessment

Refers to the nomogram in Graph 13.

Mark the bridge span on Col. A and the total crown thickness d + h (ring + fill) on Col. B. Line through these points to Col. C, and read off the provisional axle loading assessment in tons, which is then modified by the following factors.

6. Modifying Factors

The following modifying factors must now be assessed:-

(a) Span/Rise Factor

Flat arches are not so strong under a given loading as those of steeper profile, and the provisional assessment must, therefore, be adjusted. A span/rise ratio of 4 and less is assumed to give optimum strength and has a factor of 1. When the span/rise ratio is greater than 4, reference shall be made to the graph in Fig. 7 which gives the appropriate factor for the different ratios.

SPAN/RISE FACTORS FOR MASONRY ARCH BRIDGES

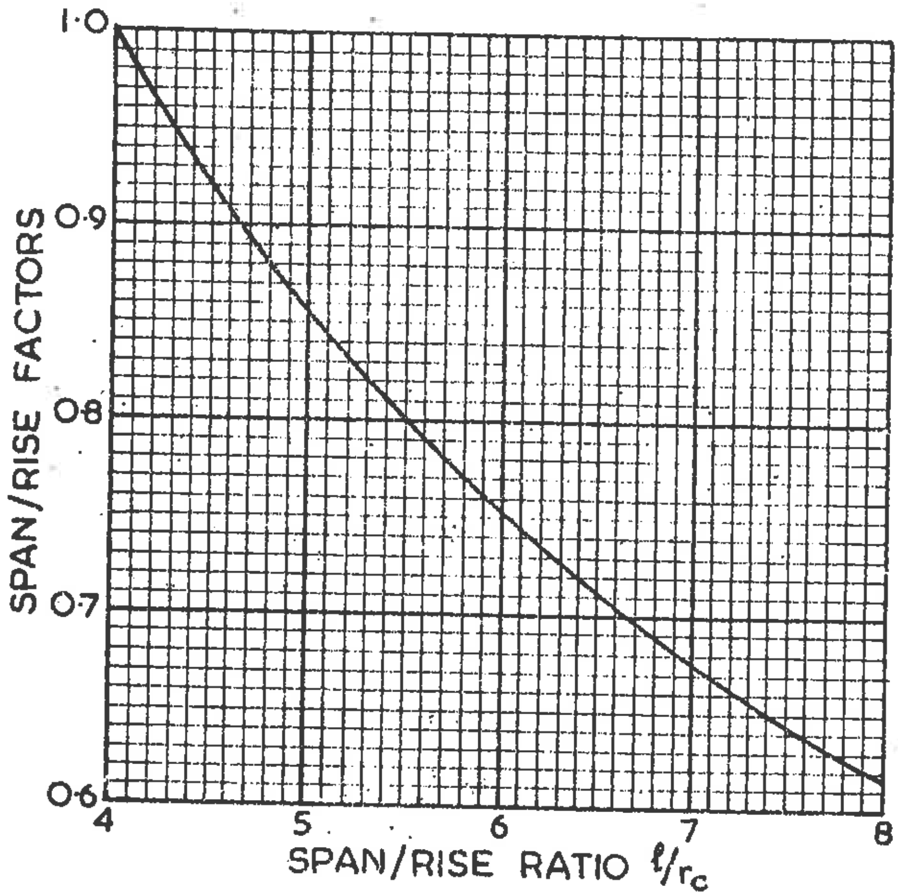


FIG.7.

(b) Profile Factor

There is evidence that elliptical arches are not so strong as segmental and parabolic arches of similar span/rise and ring thickness. The ideal profile has been taken to be parabolic and for this shape the rise at the quarter points, $r_q = \frac{3}{4} r_c$

Where r_c is the rise at the crown.

The profile factor for ratios of r_q/r_c less than or equal to ¾ shall be taken to be unity, and for ratios greater than ¾ shall be calculated from the expression -

$$F_p = 2.3\left[\frac{r_c - r_q}{r_c}\right]^{0.6}$$

For convenience this has been plotted in Fig.8.

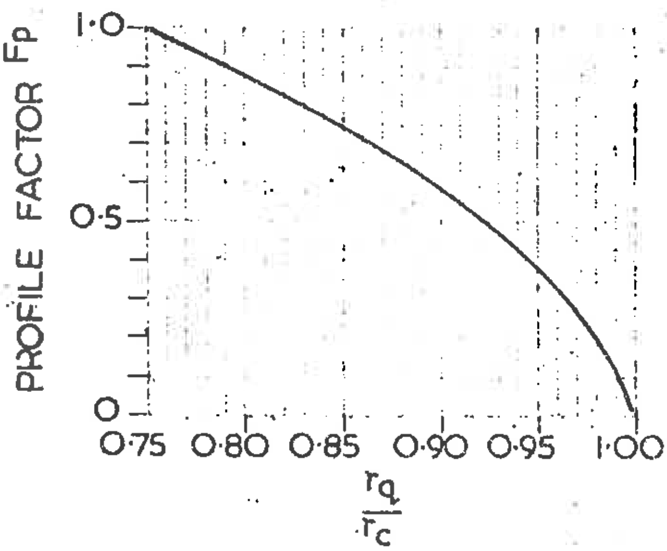


FIG.8.

(c) <u>Material Factor</u>

To determine this factor, the following points must be noted:-

(i) The material used for the ring and its thickness.

(ii) The type of construction - i.e., are the voussoirs in courses or laid at random?

(iii) The condition of the material - i.e., is there a lot of spalling, and are the voussoirs sound or are they deteriorating due to weathering?

(iv) The type of filling above the arch and its thickness at the crown

Arch Ring	Ring Factor (Fr)
Granite, whinstone and built-in-course masonry, with large shaped voussoirs	1.5
Concrete or engineering bricks (see note)	1.2
Limestone, good random masonry and building bricks in good condition	1.0
Masonry of (any kind) or brickwork in poor condition (many voussoirs flaking or badly spalling, shearing, etc.). Some discretion is permitted if the dilapidation is only moderate	0.7

NOTE: Concrete arches will normally be of relatively recent construction and their assessment should be based on the design calculations if these are available.

TABLE IV

Filling	Fill Factor (F_r)
Concrete slab or saddle (see note 1)	1.0
Grouted materials (other than those with a clay content)	0.9
Well compacted materials (see note 2)	0.7
Weak materials evidenced by tracking of the carriageway surface	0.5

NOTE 1:- The fill factor for concrete is less than the ring factor to allow for possible lack of abutment to the concrete saddle and bond to the arch. It may be increased at the engineer's discretion to a maximum of 1.2 if the details of the saddling are known to be satisfactory.

NOTE 2 When assessing an arch bridge for Construction and Use Traffic, unless details of the fill are known or there is evidence of weakness from the condition of the road surface, it is recommended that this factor be adopted. If the bridge then requires a restriction further investigation should be made to see if it may be increased.

$$\text{Material Factor } F_m = \frac{(\text{Ring factor x ring thickness})+(\text{Fill factor x fill depth})}{\text{total thickness of ring and fill at crown}}$$

$$= \frac{(F_r \times d) + (F_f \times h)}{h+d}$$

(d) Joint Factor

The strength and stability of the arch ring depend, to a large extent, on the size and condition of the joints.

Lime mortar is commonly used in bridge construction, particularly on old bridges, and, although it is softer than cement mortar, and has a lower strength, this is compensated for by better joint-filling properties and good distributing power under load.

TABLE V

Width of Joint	Width Factor (F_w)
Joints with widths up to $\frac{1}{4}$"	1.0
Joints with widths between $\frac{1}{4}$" and $\frac{1}{2}$"	0.9
Joints with widths over $\frac{1}{2}$"	0.8

Depth of Joint	Depth Factor (F_d)
Pointed joints in good condition	1.0
Unpointed joints, pointing in poor condition and joints with up to ½" from the edge insufficiently filled.	0.9
Joints with from ½" to one tenth of the thickness of the ring insufficiently filled.	0.8
Joints insufficiently filled for more than one-tenth the thickness of the ring	At Engineer's discretion.

Interpolation between these values is permitted, depending upon the extent and position of the joint deficiency.

TABLE VII

Condition of Joint	Mortar Factor (F_m)
Mortar in good condition	1.0
Loose or friable mortar	0.9

Joint factor = Width factor x depth factor x mortar factor

$$F_J = F_w \times F_d \times F_m$$

(e) Condition Factor

The estimation of the preceding factors is based on quantitative information obtainable from a close inspection of the structure, but the factor for condition of the bridge depends much more on an objective assessment of the importance of the various cracks and deformations which may be present and how far they may be counterbalanced by indications of good material and workmanship. The following notes are given as a guide.

Cracks or deformation which may have occurred soon after the bridge was built are not usually so serious as those which are recent, when they will show clean faces, possibly with loose fragments of masonry.

important point is whether the deterioration is progressive and where this is suspected, frequent careful observations may be necessary before arriving at a final assessment.

Cracks may on occasion be formed in the mortar only and it is important that cracking and joint deficiencies should not be confused with each other.

The following is a list of unfavourable and favourable circumstances, with a description of their significance:-

(i) Longitudinal cracks near the edge of the arch, signs of movement between the arch and spandrel or bulging of the spandrel, caused by the lateral spread of the fill exerting an outward force on the spandrels. This is a frequent source of weakness in old arch bridges and the proximity of the carriageway to the parapet should be taken into account when assessing its importance.

(ii) Longitudinal cracks due to differential settlement in the abutments. These are dangerous if large, because they indicate that the ring has broken up into narrower independent rings.

(iii) Lateral cracks or permanent deformation of the arch, which may be caused by partial failure of the arch or movement at the abutments. These faults may be accompanied by a dip in the parapet which may be more easily observed.

(iv) Diagonal cracks - these normally start near the sides of the arch at the springing and spread up towards the centre of the bridge at the crown. They are probably due to subsidence at the sides of the abutment, and if extensive may indicate that the bridge is in a dangerous state.

(v) Cracks in the spandrel walls near the quarter points frequently indicating flexibility of the arch ring over the centre half of the span.

(vi) Movement or cracking of the wing walls is another common source of weakness in old bridges and occurs for similar reasons to (i) above.

(vii) Where the bridge consists of multi-span arches, and the strength of intermediate piers is in doubt the structure should be closely examined for cracks or deformation arising from this cause.

(viii) Radial displacement of individual stones or bricks especially near the crown when there is little cover. Displacement may be due to uneven masonry projecting above the barrel and being subjected to concentrated loads, or a hard spot such as a pipe flange bearing directly on the arch. The damage is usually localised and not serious if dealt with before it has progressed too far.

(ix) An arch which is constantly wet or shows signs that damp often penetrates is likely to have suffered deterioration from this cause and a reduction of the condition factor may be warranted although cracks and deformations are absent.

(x) Some local damage may be offset by evidence that the structure was built with good materials and workmanship. Such evidence will be:-

(a) Durable masonry set in its correct bed.

(b) Well shaped durable bricks.

(c) Correct bonding of brickwork or masonry with regular and narrow joints.

(d) Original documents showing liberal haunching at the abutments and a good specification.

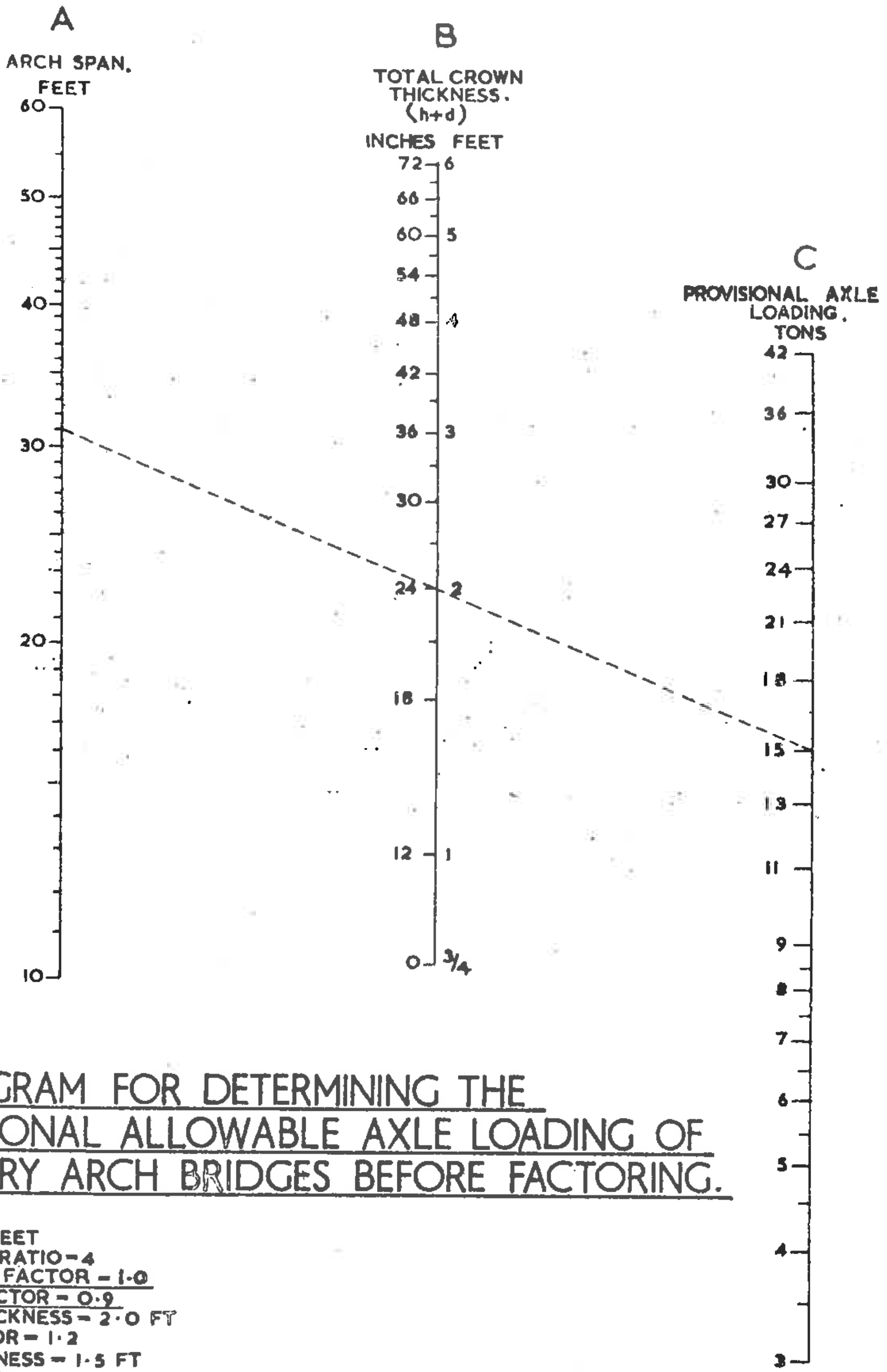
(xi) The width of an arch should not per se, influence its carrying capacity for C & U vehicles, as the theoretical work from which the M.E.X.E. Assessment was adapted, considered a lane to be carried by an 8' width of structure. Where however a carriageway is close to its edge, an arch is particularly vulnerable to the damage described in (i) above.

When the condition of the bridge has been critically examined, the Engineer should give a quantative estimate of its condition which will lie between 0 and 1.0. In arriving at this factor the Engineer should dissociate the "condition factor" from the "material factor" and "joint factor" as these are dealt with separately as described earlier. Where the condition factor is less than 0.4 or the final allowable axle load is less than 5 tons immediate consideration should be given to the repair or reconstruction of the bridge.

Should, for any reason, there be disagreement between the Bridge Owner and the Highway Authority over the value of the condition factor to be taken for an arch bridge, the disagreement should be referred to Bridges Engineering Department of the Ministry of Transport.

9. Application

The span/rise profile, material, joint, and condition factors should be applied together with the provisional assessment obtained as in para. 7 in order to determine the final axle load to which the bridge should be restricted, the calculated value being rounded up or down to the nearest ton.



NOMOGRAM FOR DETERMINING THE PROVISIONAL ALLOWABLE AXLE LOADING OF MASONRY ARCH BRIDGES BEFORE FACTORING.

EXAMPLE:-

SPAN – 31 FEET
SPAN/RISE RATIO – 4
∴ SPAN/RISE FACTOR – 1·0
PROFILE FACTOR – 0·9
CROWN THICKNESS – 2·0 FT
RING FACTOR – 1·2
RING THICKNESS – 1·5 FT
FILL FACTOR – 0·7
FILL DEPTH – 0·50 FT
∴ MATERIAL FACTOR – 1·07 — $\frac{1 \cdot 2 \times 1 \cdot 5 + 0 \cdot 7 \times 0 \cdot 5}{2 \cdot 0}$
WIDTH FACTOR – 0·9
DEPTH FACTOR – 0·9
MORTAR FACTOR – 1·0
JOINT FACTOR – 0·81 — 0·9 X 0·9 X 1·0
CONDITION FACTOR – 0·6

THE PROVISIONAL AXLE LOADING FOR AN ARCH, 31 FT SPAN AND CROWN THICKNESS OF 2·0 FT IS, FROM THE NOMOGRAM, 15 TONS.
ALLOWABLE AXLE LOAD = 15 X 1·0 X 0·9 X 1·07 X 0·81 X 0·6 = 7·42 TONS
∴ APPLY A 7 TON AXLE LOAD RESTRICTION TO THE BRIDGE.

Technical Memorandum (Bridges) No.B.E.4
The Assessment of Highway Bridges for Construction and Use Vehicles
Explanatory Note

Part III Clause 5(e)

Due to lack of clarity in fig.6 the precise wording of this sub-clause has not always been appreciated. The dimension 'h' is to be taken as 'the average depth of fill between the road surface and the arch ring at the crown' and not 'the average depth of fill between the road surfacing and the arch ring at the crown' (i.e. the depth of fill is to include the road surfacing).

The diagram should be amended as shown below.

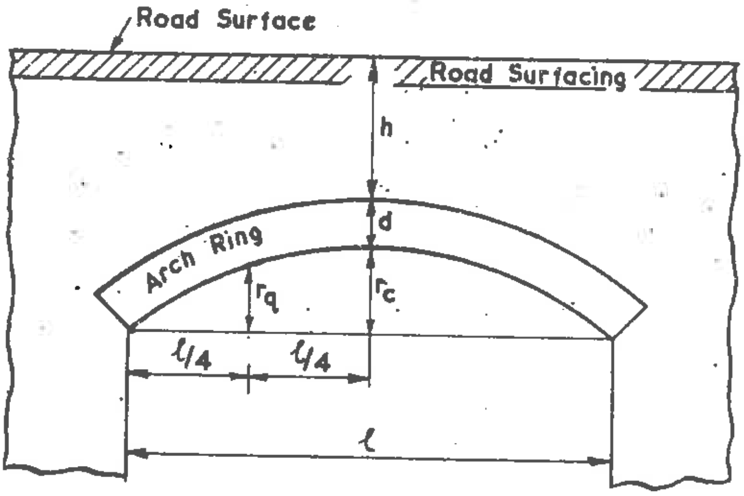


FIG. 6.

L. G. Deuce
Assistant Chief Engineer

Bridges Engineering Division
Ministry of Transport
St. Christopher House
Southwark Street, S.E.1.

13th February, 1969.