

Category 1 Standard

S1061 Civil Engineering – Bridges and Structures Assessment Standard

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1 Purpose

- 1.1 The purpose of this Standard is to set down the requirements for the assessment of bridges and structures assets owned by London Underground, and which are not known to have been designed to loading standards equal to or in excess of current standards, and/or those structures thought to have a reduced loading capacity as a result of deterioration, damage or increased imposed loading.
- 1.2 This standard shall not be used for the assessment of structures carrying the public highway. Section 117 of the 1968 Transport Act sets down London Underground's duty in respect of bridges carrying the public highways and owned by LU. This duty is
- That in constructing or reconstructing a bridge that on completion will belong to LU to ensure that it has the required load bearing capacity.
 - To maintain and improve or strengthen any bridge belonging to LU so that it has the required load bearing capacity.
- The capacities were not specified in the Act but published later in Statutory Instrument No 1705 entitled Highways England and Wales The Railway Bridges (Load – Bearing Standards)(England and Wales) Order 1972. This Order set two standards, an assessment standard defined in DoT Technical Memorandum BE4 for existing bridges and a design load (HA loading) set down in BS153 for new bridges. In 1984 the DoT issued an assessment code for highway bridges (BD21/84). This has since been revised.
- Therefore bridges built before 1968 shall first be assessed to BE4 and then re-assessed to BD21. Bridges built after 1968 if requiring assessment shall be assessed to BD21.
- 1.3 Road bridges which do not carry a public road (Accommodation and Occupation bridges) need only be assessed for 3 tons GVW for bridges built before 1903 and 5 tons GVW for bridges built after 1903 unless there is a specific legal agreement to carry a higher load. Thus BD21/01 shall be used for assessment purposes, but only to determine the structures' capability of carrying 3 ton or 5 ton GVW. Accommodation and Occupation bridges must always have their safe load capacity prominently displayed on signs at each end of the bridge.
- 1.4 There is no requirement in statute nor in any National standard to assess the capacity of highway bridge parapets. London Underground has carried out an assessment of the containment capacity of its highway bridge parapets and this is reflected in the ACAC concern XGEN803 "Overbridge parapets that do not meet type P6 requirements". No further containment assessment is required.

2 Scope

- 2.1 This Standard applies to bridges and structures assets owned by London Underground.

3.1 General Requirements

- 3.1.1 This standard shall be used for the assessment of the following structures:
- a) underbridges constructed of steel, concrete, wrought iron and cast iron as well as brick arches. The obsolete Department of Transport publication 'Requirements for Passenger Lines and Recommendations for Goods Lines of the Ministry of Transport in regard to Railway Construction and Operation' forbade the use of cast iron in railway bridges. If cast iron underbridges are encountered, they shall be assessed using BD21 but modified to recognise RL or RU loading.
 - b) any railway operational building or station structure, but not CMRs, IMRs or the like or modular buildings.
 - c) brick and concrete subsurface tunnels carrying building loads.

- d) Footbridges, cable bridges and gantries constructed of steel, concrete, wrought iron and cast iron as well as the masonry portions of these structures.
 - e) Boundary walls.
 - f) Retaining walls i.e. 'gravity walls', cantilever reinforced concrete, gabion and embedded retaining walls' e.g. sheet piles and contiguous bored piles.
- 3.1.2 Analytical Assessments shall be carried out to achieve the following objectives:
- a) to determine the degree to which assets remain safe and fit to fulfil the structural element of their Required Duty in their current environment and Condition;
 - b) to provide the information on structural capacity of the assets in a consistent and accurate manner;
 - c) to provide information necessary for the managed Maintenance strategy of the assets;
 - d) to confirm that assets are safe for railway operations and for public and Outside Party use; and
 - e) to provide the appropriate information on assets to meet the requirements for the Asset Condition Assessment.
- 3.1.3 If it can be shown by reference to documents that a structure has been designed to current LU loading standards then there is no requirement to carry out an assessment unless it is thought to have a reduced load capacity due to deterioration, damage or increased dead load.
- 3.1.4 Assets shall be subject to Analytical Assessment in accordance with the whole life Maintenance strategy programme for the asset and when:
- a) a change of Required Duty is proposed or has occurred; or
 - b) an inspection has shown a change in Condition that may indicate a reduction in the structural capacity of the asset; or
 - c) an Incident occurs or is reported that may indicate a reduction in the structural capacity of the asset.
- 3.1.5 Structures assessed using this standard shall use the limit state format with partial safety factors although brick arches, cast iron, timber and filler joist slabs are exceptions. Cast iron, timber structures and filler joist slabs shall be assessed on a permissible stress basis.
- 3.1.6 Limit state requirements are applicable to the assessment of masonry arch bridges. However, unless a suitable rigorous method of analysis is used conforming to the principles of clause 3.2.5 – 3.2.7, arches will be assessed by a graphical method as described in 3.2.22.
- 3.1.7 **This standard uses the following British Standards as a basis for assessment despite the fact that they are not now supported by the BSI: CP3 Chap V, BS5268, BS 5400, BS5628, BS5950, BS6399, BS8002, BS8110. These British Standards are “amendment specific” and reference must be made Section 6.1 to ensure that the correct amendment is being used.**
- 3.1.8 No assessment is required for the serviceability limit state.
- 3.1.9 Inspection for assessment**
- 3.1.9.1 The assessment of a structure to determine its load carrying capacity involves not only analysis and calculations but also the inspection of the structure concerned. Prior to the commencement of the assessment, the structure shall be inspected by the assessor to verify the form of construction, the dimensions of the structure and the

nature and condition of the structural components. This inspection shall cover not only the condition of individual components, but also the condition of the structure as an entity, and especially noting any signs of distress and their cause. The inspection shall comply with the standard S1060 together with the additional requirements set down in the sections relating to specific structural types.

3.2 Underbridge Assessment

- 3.2.1 Three categories of live loading described in table 1 below are specified for the assessment of bridges carrying railway tracks vested in LUL. The particular live loading category to be adopted for each bridge assessment shall be identified from that table.

Bridge live loading category	Railway lines carried	Live loading
1	Metropolitan Line - Neasden to Amersham and beyond (including the Chesham and Watford branches) Jubilee Line - Neasden to Wembley Park District Line - E. Putney to Wimbledon Railtrack tracks Moorgate to Farringdon	BS 5400: Part 2 RU loading BS 5400: Part 2 RU loading BS 5400: Part 2 RU loading BS 5400: Part 2 RU loading
2	All lines not in category 1	Clause 3.2.13 RL loading
3	Service Walkways forming part of the superstructure a) Deck	1kN concentrated loads at 1m centre for the assessment of the walkway. The walkway shall be considered to be unloaded in the assessment of the bridge structure
	b) Handrails	0.74kN/m applied horizontally at the level of the top handrail.

Table 1 Categories of live loading

- 3.2.2 The procedures given in this section are not of general application for the assessment of sub-structure, foundations, wing walls and spandrel walls: their assessment shall be based on qualitative judgement of the information obtained from their inspection, in accordance with the requirements of standard S1060. The exception to this is those bridges where the superstructure is supported on one or more intermediate supports comprising reinforced concrete or steel piers, trestles, or columns, in which case the capacity of each support shall be assessed.

3.2.3 Inspection for loading

- 3.2.3.1 The structure shall be inspected to determine the density and dimensions needed to calculate the nominal loads Q_k . Care shall be taken to obtain an accurate estimate of dead and superimposed loads by undertaking a geometric survey of the structure to verify record drawings and documents where these exist.



- 3.2.3.2 Loads due to ballast are very significant and measurements shall be taken in order to enable calculations of its dead-weight to be undertaken. It should also be noted that the presence of services and ducts on the bridge, where indicated in record and as-built drawings, is no guarantee that they exist or are in the locations shown. The presence of these shall be verified where they will affect the results of the assessment calculations.
- 3.2.3.3 In addition to the above, the clear dimensions between the soffits of the bridges and the highest highway or rail level shall be measured and also the position of the rails relative to the structure. Levels shall be related to the Newlyn Ordnance datum. The horizontal alignment of the tracks, when curved on the structure, shall be determined to permit the calculation of centrifugal loads.

3.2.4 Inspection for resistance

- 3.2.4.1 The structure shall be inspected to record all the parameters needed to determine the strength of members and elements, including possible deficiencies, e.g:
- a) cracks, damage caused by vehicle impact, deformations;
 - b) spalling of concrete, exposed reinforcement;
 - c) corrosion, and material loss in load carrying members through corrosion;
 - d) loosening of joints (rivets, bolts or other fixings);
 - e) repairs, partial renewals, strengthening and the efficiency of such operations.
 - f) anomalies in bridge behaviour under the passage of trains;
 - g) deterioration and settlement affecting the supports and bearings.
- 3.2.4.2 The inspection shall provide confirmation of the information obtained from drawings and documents, particularly:
- a) dimensions of internal sections that may not be related to external features;
 - b) reduction in strength due to services, laid through, or near, the structure.
- 3.2.4.3 With regard to buried members, e.g. trough decks, where there is doubt on the above parameters, excavation of trial holes shall be considered.
- 3.2.4.4 The inspections shall be written up on the pro-formas set down in standard S1060.

3.2.4.4 Brick arch bridges

- 3.2.4.4.1 External fabric shall always be inspected. Investigation into the construction shall be necessary where the strength of the bridge is in doubt.

The arch barrel shall be inspected to record all the information needed to determine the loading and resistance in accordance with 3.2.22.6.4. In particular, the following information shall be obtained:

- a) nature and condition of the brickwork or concrete including the location and extent of any crushing;

- b) thickness of joints and depth of mortar missing;
- c) condition of mortar;
- d) presence of cracks - their width, length, position and number;
- e) location of any displaced voussoirs;
- f) deformation of the arch barrel from its original shape;
- g) any additional strengthening of rings, or saddling.

The inspection shall provide verification of the information already obtained; for example, many viaducts will consist of arches in which the spandrels have been partly filled or totally filled with concrete. Where there is any doubt about the conditions, a site investigation shall be made, including the digging of trial holes.

Parapets and spandrel walls

3.2.4.4.2 Parapets and spandrel walls shall be inspected to obtain evidence of any defects and their extent recorded, e.g.:

- a) tilting, bulging or sagging;
- b) lateral movement of parapet or spandrel wall relative to the face of the arch barrel;
- c) lateral movement of parapet or spandrel wall accompanied by longitudinal cracking of the arch barrel;
- d) weathering and lack of pointing;
- e) cracking, splitting and spalling.

Abutments, piers, foundations and wing Walls

3.2.4.4.3 Inspection of arch bridge abutments, piers, foundations and wing walls shall be in accordance with 3.1.4.

3.2.4.5 Sub-structures, foundations and wing walls

General

3.2.4.5.1 All accessible parts of the substructures, foundations and wing walls shall be examined and any defects noted. For sub-structures founded in water, underwater inspection may be required to determine their condition.

Bearings

3.2.4.5.2 The presence or otherwise shall be noted and, if present, they shall be inspected to obtain information on the following:

- a) general condition ;
- b) any binding or jamming, looseness or reaching the limit of movement;
- c) condition of seating, bedding and plinth;

- d) whether correct operation of the bearing is prevented or impaired, e.g. by structural members built into abutment or pier.

Piers, bank seats, abutments and wing walls

3.2.4.5.3 The inspection shall obtain information on whether the following defects are present and, if so, their extent:

- a) tilting and rotation, in any direction;
- b) rocking;
- c) cracking, splitting and spalling;
- d) erosion beneath water level;
- e) weathering and other material deterioration, including lack of pointing for brickwork;
- f) growth of vegetation;
- g) lack of effective drainage;
- h) internal scour, and leaching of fill;
- i) settlement of fill;
- j) scour of supporting material.

3.2.4.5.4 Scour inspections mentioned in clause 3.2.4.5.3 j) shall only be carried out if a scour assessment carried out in accordance with clause 3.2.26 gives a risk priority greater than 13.

3.2.5 Assessment load values

Assessment loads

3.2.5.1 The assessment loads, QA, are determined from the nominal loads, QK, according to the equation:

$$QA = \gamma_{fl} \cdot QK$$

Where γ_{fl} is a partial factor for each type of loading as given in table 2.

Loading	Material	γ_{fl}^*
Dead	Cast iron	1.10
	Steel, wrought iron	1.05
	Concrete, stone and brick, masonry, timber	1.15
Superimposed Dead	Sleepers, rails, and fixings	1.05
	All types of ballast	1.10
	All other filling, make up materials, spandrel walls	1.20
Live load		1.25

Centrifugal loads		1.25
Nosing		1.25
Lurching		1.25
* Except in cases where γ_{fl} causes a less severe effect than if a γ_{fl} of 1.0 had been adopted, in which case values of 1.0 shall be adopted.		

Table 2 Partial factors for loads γ_{fl}

Load combinations

- 3.2.5.2 Nominal dead and superimposed dead loads shall be combined with the live loads using the partial factors in table 2. When other loads not specified in this standard are considered to have a significant effect on the result of the assessment, reference shall be made to BS 5400: Part 2 for details of the relevant loads, load combinations and partial factors.

Assessment load effects

- 3.2.5.3 The assessment load effects, S_A , shall be obtained from the assessed loads by the relationship:

$$S_A = \gamma_{f3} \text{ (effects of QA)}$$

$$= \gamma_{f3} \text{ (effects of } \gamma_{fl} \cdot QK)$$

where γ_{f3} is a factor that takes account of inaccurate assessment of the effects of loading such as unforeseen stress distribution in the structure, inherent inaccuracies in the calculation model, and variations in the dimensional accuracy from measured values.

- 3.2.5.4 The effects of the assessment loads are to be obtained by the use of the appropriate form of analysis in accordance with the requirements of 3.2.19. For the purpose of this standard, the value of γ_{f3} shall be taken as 1.1.

3.2.6 Assessment of Resistance

3.2.6.1 Calculated resistance for Underbridges

The calculated resistance, R_A^* , determined from material strengths and measured section properties, shall be calculated from the following expression:

$$R_A^* = \text{function} \left(\frac{f_k}{\gamma_m} \right)$$

where f_k is the characteristic (or nominal) strength of the material as given in 3.2.10 and γ_m is a partial factor for material strength as given in table 3.

For steel and wrought iron construction, the expression may be modified as:

$$R_A^* = \frac{1}{\gamma_m} \cdot \text{function} (f_k)$$

Where f_k is the characteristic (or nominal) strength of the material given in 3.2.10 and γ_m is a partial safety factor for material strengths given in the table 3

Material	Structural component and material	γ_m
Steel	All components except those listed below	1.05
	Bending resistance of beams related to the limiting compressive stress	1.2
	Buckling resistance of stiffeners	1.2
	Fasteners in tension	1.2
	Fasteners in shear	1.2
	Welds	1.1
	Friction capacity of HSFG bolts	1.3
Wrought iron	All	1.2
Cast iron	All components	1.0
Reinforced Concrete	Concrete	1.5
	Reinforcement	1.15
Masonry	All components	2.5

Table 3 Values of γ_m - partial factor for material strength

3.2.6.2 The strength of the sections shall be determined in accordance with the requirements of section 3.2.23. Wherever possible, the existing sound thickness after allowing for corrosion and cracking of the critical components shall be measured and used in determining RA*.

Condition factor

3.2.6.3 If the measurement of sound thickness is not possible, or if there are other uncertainties in the determination of resistance, a condition factor, FC, shall be estimated to account for any deficiencies that are noted in the inspection (see section 8) but not allowed for in the determination of calculated resistance RA*. The value of FC shall represent, on the basis of engineering judgement, an estimate of any deficiency in the integrity of the structure. This may relate to a member, a part of the structure, or the structure as a whole. The value taken for FC shall not be greater than 1.0.

3.2.7 Verification of Structural Adequacy

Structures shall be deemed to be capable of carrying the assessment load when the relationship shown in figure 1 is satisfied.

$RA \geq SA$	Equation 1
$\left(\frac{fk}{\gamma_m} \right) \geq \gamma_{f3} \text{ (effects of } \gamma \text{ fl.QK)}$	Equation 2
i.e. FC.function	
For steel and wrought iron construction, the relationship may be rearranged as follows:	
$\frac{FC}{\gamma_{f3} \gamma_m} \cdot \text{function (fk)} \geq \text{(effects of } \gamma \text{ fl.QK)}$	Equation 2b

Figure 1 Capability of structure to carry assessment load.

3.2.8 Fatigue Assessment

- 3.2.8.1 Fatigue assessments on steel structures shall be carried out in accordance with BS 5400: Part 10, amended as described in 3.2.8.2 to 3.2.8.7. The purpose of the assessment is to determine the expired fatigue life of the structure in years
- 3.2.8.2 The loads to be considered shall be as specified in the relevant parts of this standard, with the following exceptions:
- a) only RL loading shall be considered, even on lines where rolling stock from other train operating companies is permitted to pass;
 - b) For fatigue assessment purposes, only 30% of RL loading shall be applied to the structure.
- 3.2.8.3 Where flanges are corroded, all corrosion products shall be removed and the minimum reduced net area shall be calculated at all sections. The maximum stress range in the member shall then be calculated. Figure 15 in BS 5400: Part 10 shall be used, assuming that the stress raising effect of the corroded surface is no greater than that of the holes.
- 3.2.8.4 Where webs are corroded, a procedure similar to that for flanges shall be adopted, but a reduced area along a horizontal section of length shall be calculated, assuming the same stress raising effect as in 9.6.3.
- 3.2.8.5 In calculating factor K4 (Table 5 in BS 5400 Part 10), the following traffic movements shall be used and the total annual tonnage given in Table 16 of BS 5400: Part 10 shall be adjusted in direct proportion to the following:
- a) Hammersmith and City Line - 108,50 trains per annum.
 - b) District and Piccadilly Lines - Hammersmith to Acton Town - 130,000 trains per annum;
 - b) Northern Line - 195,000 trains per annum;
 - c) Metropolitan Line - Rickmansworth to Amersham - 10,700 trains per annum.
- 3.2.8.6 For all tracks other than those defined in 3.2.8.5, the most recently available actual annual traffic figures shall be obtained. It shall be assumed that the most recently available annual traffic movements have occurred every year since the construction of the bridge carrying the track.
- 3.2.8.7 The number of cycles of loading shall be calculated from Table 3 of BS 5400: Part 10. The number of live load cycles referred to in Groups 1 and 2 shall be added to revised numbers of cycles in Groups 3, 4, 5 and 6 to reflect the 30% reduction of RL loading and these shall also be adjusted to reflect the specified or actual numbers of trains passing, as described in 3.2.8.5 and 3.2.8.6 respectively.

3.2.9 Properties of materials

Unit weights of materials

- 3.2.9.1 For initial assessment, the appropriate values of the material properties given in Attachment 1 shall be used.

Co-efficients of expansion

- 3.2.9.2 For initial assessment the coefficient of linear thermal expansion given in Attachment 2 shall be used.

3.2.10 Strengths of materials (function f_k)

General

- 3.2.10.1 For the initial assessment, the characteristic strengths of materials shall be taken as specified in 3.2.10.2 to 3.2.10.4. Testing should normally be carried out only if the initial assessment is considered inadequate or if there is some doubt about the nature and quality of the materials. The strength values obtained from a limited number of tests should be considered as only an indication of whether the characteristic strength values given in 3.2.10.2 to 3.2.10.4 are applicable to the material present in the structure.

Steel

- 3.2.10.2 In the absence of definite information, a characteristic strength of 230 N/mm² shall be assumed for steel produced before 1955. Some of the pre-1922 steels were of poor quality and shall be closely inspected for laminations, inclusions and deformities. Since about 1955, steel has been available in various grades with different levels of yield stresses, hence it is essential to identify the particular grade and specification of the steel on the structure from available documents. From this information and reference to the specification, it shall be possible to determine yield stress that can be used for assessment.

Concrete

- 3.2.10.3 Pre-1939 concrete shall be assumed to have a characteristic strength not greater than 20 N/mm². Subsequent to this date, available documentation shall be consulted and the recommendations in BS 5400: Part 4 with respect to gain in strength with age, used to determine the characteristic strength.

Wrought iron

- 3.2.10.4 The quality of wrought iron may depend upon where and when it was made and its strength can vary considerably. It must always be carefully examined for lamination, inclusions and deformities. For assessment purposes, the characteristic yield stress shall be taken as 220 N/mm².

3.2.11 General

- 3.2.11.1 Structures shall be assessed to the loading requirements given in this section. Assessment loading will generally be limited to the application of dead and superimposed dead loads and type RL or RU live loads - the latter two consisting of uniformly distributed loads together with concentrated loads illustrated in Figures 2 & 3. These loadings are equivalent static loadings and shall be multiplied by the dynamic factors given in 3.2.14 to allow for impact, oscillation and other dynamic effects, including those caused by track and wheel irregularities.
- 3.2.11.2 All loads specified in this section are nominal loads and shall be multiplied by the appropriate partial factors given in table 2.
- 3.2.11.3 The type RL and RU, UDL and axle loads do not model the effect of rail vehicles on short span masonry arches. This type of structure shall be assessed using the method for masonry arches.

3.2.12 Nominal dead load

The nominal dead load and nominal superimposed dead load shall be derived having regard to attachment 2.

3.2.13 Nominal live loads

General

- 3.2.13.1 The RU and RL loading covers the ranges of rail vehicles at present authorised to travel on tracks vested in LUL. The live loadings shall be applied to the superstructure and its elements in accordance with the loading requirements of BS 5400: Part 2. The assessment shall include allowances for lurching, nosing and longitudinal forces and, where the railway tracks are curved in plan, the centrifugal effects.

Type RL loading

- 3.2.13.2 Nominal type RL loading consists of a single 200kN concentrated load positioned to generate the most severe effect on the element under consideration, coupled with a uniformly distributed load of 50kN/m for loaded lengths up to 100m. For loaded lengths in excess of 100m, the distributed nominal loads shall be 50kN/m for the first 100m and shall be reduced to 25kN/m for lengths in excess of 100m, as shown in figure 2.

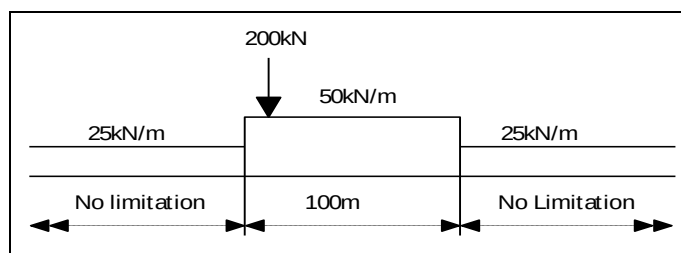


Figure 2 RL loading

- 3.2.13.3 Alternatively, two concentrated nominal loads, one of 300kN and the other of 150kN, spaced at 2.4m intervals along the track, shall be used on deck elements where this gives a more severe condition. These two concentrated loads shall be deemed to include dynamic effects.

Type RU loading

- 3.2.13.4 Nominal type RU loading consists of four 250kN concentrated loads, preceded and followed by a uniformly distributed load of 80kN/m. The arrangement of this loading is as shown in figure 3.

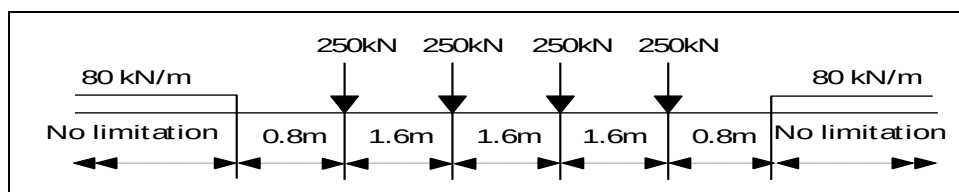


Figure 3 RU Loading

Reduced loading

- 3.2.13.5 Should the bridge fail to meet the requirements of the standard loading then its capacity as a percentage of the standard loading shall be calculated so that reduced loading can be determined.

Service walkways

- 3.2.13.6 Components of service walkways shall be assessed separately to carry the live loading given in table 1. However, this live loading shall not be considered in the assessment of the bridge structure.

3.2.14 Dynamic effects

General

3.12.14.1 The factors described in 3.2.14 shall be applied only to the vertical components of RL or RU loading - not to centrifugal or nosing effects.

RL loading

3.12.14.2 The nominal RL loading shall be multiplied by a dynamic factor. This factor shall be 1.2 for ballasted tracks and 1.4 for rail bearers and single track cross girders with unballasted track.

RU loading

3.12.14.3 The nominal RU loading shall be multiplied by the factors given in table 4. It shall be applied to ballasted track, railbearers and single track cross girders with unballasted track.

Dimension L (M)	Dynamic Factor for evaluating	
	Bending moment	Shear
Up to 3.6	2.0	1.67
From 3.6 to 67	$0.73 + 2.16/\sqrt{(L-0.2)}$	$0.82 + 1.44/\sqrt{(L-0.2)}$
Over 67	1.0	1.0

Table 4 RU loading – dynamic effects

3.12.14.4 In calculating the dynamic factor, L shall be taken as the length (in metres) of the influence line for deflection of the element being considered. For unsymmetrical influence lines, L is twice the distance between the point at which the greatest ordinate occurs and the nearest end point of the influence line. In the case of floor members, 3m shall be added to the length of the influence line as an allowance for load distribution through the track. The values given in table 5 shall be used if appropriate.

Member	Dimension L (m)
Main girder Simply supported	For 2,3,4,5 and more spans $1.2, 1.3, 1.4, 1.5 \times$ mean span, as appropriate but at least the greatest span.
Floor members, Simply supported rail bearers.	Cross girder spacing plus 3m.
Cross girders loaded by simply supported rail bearers.	Twice the spacing of cross girders plus 3m.
End cross girders.	4m
Cross girders loaded by continuous deck elements and any element in a continuous deck system.	The lesser of the span of the main girders or twice the main girder spacing.

Table 5 Dimension L used in calculating the dynamic factor

3.2.15 Centrifugal effects (RL and RU Loading)

Where the track on the bridge is curved, centrifugal effects of moving loads on the structure shall be assessed. All tracks on the structure shall be considered occupied. The nominal centrifugal load F_a shall be taken to act outwards in a horizontal direction at a height of 1.80m above rail level calculated from the following formula:

$$F_a = \frac{P(v + 10)^2 \times f}{127r}$$

Where

P' = The static equivalent uniformly distributed load for bending moment when assessing for RU loading; for RL loading, a distributed load of 40 kN/m multiplied by L is deemed sufficient.

r = Radius of curvature (in m)

v_t = Maximum speed in km/h

f = Reduction factor depending on speed v_t and length L given by the formulae where:

L = Length in metres of the loaded portion of the curved track

a) for $L > 2.88\text{m}$ and $v_t > 120\text{km/hr}$

$$f = 1 - (v_t - 120/1000) \times (814/v_t + 1.75) \times (1 - \sqrt{2.88/L})$$

b) for $L < 2.88\text{m}$ or $v_t < 120\text{km/hr}$

$$f = 1$$

3.2.16 Nosing - (RL and RU loading)

- 3.2.16.1 An assessment allowance for nosing shall be taken as 100kN acting horizontally at the top of the rail perpendicularly to the track in either direction and in the most unfavourable position. It shall be assumed that centrifugal loads and this nosing load act simultaneously.
- 3.2.16.2 The vertical effects of this load on secondary elements such as cross girders and rail bearers shall be considered.
- 3.2.16.3 For elements supporting more than one track, a single load as specified is deemed sufficient.

3.2.17 Lurching

RL loading

- 3.2.17.1 To allow for lurching, 0.56 of the track load shall be considered acting on one rail concurrently with 0.44 of the track load on the other rail. This redistribution need only be taken into account on one track where members support two tracks. Lurching may be ignored in the case of elements that support load from more than two tracks.

RU loading

- 3.2.17.2 The dynamic factor applied to RU loading takes into account the effects of lurching and the load to be considered acting on each rail shall be half the track load.

3.2.18 Longitudinal loads

- 3.2.18.1 The effects of loads due to traction and braking shall be assessed by applying the nominal longitudinal loads given in table 6. These loads shall be considered to act at rail level in a direction longitudinal to the tracks.
- 3.2.18.2 For ballasted track, one third of the longitudinal loads shall be assumed to be transmitted by the track to resistances outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on or within 18m of either end of the bridge.

- 3.2.18.3 Structures and elements carrying single tracks shall be assessed to carry the larger of the two loads produced by traction and braking in either direction parallel to the track.
- 3.2.18.4 Where a structure or element carries two tracks, both tracks shall be considered as being occupied simultaneously. Where the tracks carry traffic in opposite directions, the load due to braking shall be applied to one track and the load due to traction to the other. Structures and elements carrying two tracks in the same direction shall be subjected to braking or traction on both tracks, whichever gives the greater effect.
- 3.2.18.5 Where elements carry more than two tracks, longitudinal loads shall be considered to be applied to two tracks only.

RL	Traction (30% of load on driving wheels)	Up to 8 From 8 to 30 From 30 to 60 From 60 to 100 Over 100	80 10 300 5 500
	Braking (25% of load on braked wheels)	Up to 8 From 8 to 100 Over 100	64 8 800
RU	Traction (30% of load on driving wheels)	Up to 3 From 3 to 5 From 5 to 7 From 7 to 25 Over 25	150 225 300 24 (L-7)+300 750
	Braking (25% of load on braked wheels)	Up to 3 From 3 to 5 From 5 to 7 Over 7	125 187 250 20(L-7)+250

Table 6 Nominal longitudinal loads

3.2.19. Analysis of structure

3.2.19.1 Assumptions

Methods of analysis shall be in accordance with the principles set out in BS 5400: Part 1. Structures shall be modelled as realistically as possible and whatever approach is adopted for representing member stiffnesses, it shall be used consistently throughout the structure. Elastic methods of analysis are acceptable as safe solutions for the ultimate limit state.

3.2.19.2 Effective spans

The effective span shall be taken as:

- the distance between the webs of the supporting beams, where beam or troughing is connected to the webs of other beams;
- the distance between the centre lines of the bearings, where a beam is supported by roller, rocker, step or other purpose-made bearings other than flat plates;
- the clear span between front edges of bearing surfaces plus half the combined lengths of the bearing surfaces, where reinforced concrete slabs are free ended;
- the distance between the centroids of the bearing pressure diagrams, (assuming that the reaction is distributed linearly from a maximum at the front edge of the support to zero at the back of the bearing area), for other beams, girders, longitudinal trough girders that bear onto masonry, concrete or brick. The length

of the bearing shall not be taken as greater than the depth of the beam where the support is of brick, or more than one quarter of the depth of the beam where the support is of hard material such as concrete or stone;

e) the clear span between points of contact of the member with abutments, bedstones, or front edges of bearing plates plus 150mm each end, for longitudinal troughing.

3.2.19.3 Section properties

The section properties used for the calculation of member strengths shall be based on a realistic assessment of the state of the structure. Note must therefore be taken of:

- a) losses of material in carrying elements through corrosion;
 - b) cracks, damage caused by vehicle impacts, deformation;
 - c) loosening of joints;
 - d) anomalies in bridge behaviour under the passage of trains.
- These and any other faults in either superstructure or sub-structure must be considered and due allowance made for the adverse effect in the assessment of member strength.

3.2.19.4 Live load dispersion

General

Concentrated loads applied to the rail will be distributed both longitudinally by the continuous rail and transversely by the sleeper and ballast.

3.2.19.5 Longitudinal dispersion

In the case of ballasted decks it shall be considered that two thirds of a concentrated load applied to one sleeper will be transmitted to the bridge deck by that sleeper, and the remaining one-third will be transmitted evenly by the two sleepers either side.

3.2.19.5.1 In the case of cross girders with infill concrete supporting ballasted deck and cross sleepers (including cross girders with jack arches), as an alternative to a rigorous analysis, the following may be assumed:

- a) cross girder spacing at 1.8m or more, no dispersion allowed;
- b) cross girder spacing at less than 1.8m, concentrated loads may be reduced in the ratio:

$$fr = \text{cross girder centres (m)} / 1.8\text{m}.$$

3.2.19.5.2. In the case of cross girders with continuous or vertically lapped longitudinal timbers (providing their capability to support the live loads has been confirmed and they are correctly packed to ensure uniform load application to the cross girder), as an alternative to a rigorous analysis, the following may be adopted:

- a) cross girder spacing at 1.8m or more, no dispersion allowed;
- b) cross girder spacing at less than 1.8m and with longitudinal timbers greater than 225mm deep, concentrated loads may be reduced in the ratio:
$$fr = \text{cross girder centres (m)} / 1.8\text{m};$$
- b) cross girder spacing at less than 1.5m and with longitudinal timbers less than 225mm deep, concentrated loads may be reduced in the ratio: $fr = \text{cross girder centres (m)} / 1.5\text{m}.$

c)

3.2.19.6 Lateral dispersion

3.2.19.6.1 In the case of longitudinal troughing and slabs on ballasted decks with a depth of ballast up to 150mm between the underside of the sleeper and the top of the member, the width, L_d , considered to carry the track load shall be 3m. Where the depth of ballast exceeds 150mm, the width of deck considered to carry the track load shall be 3.6m or the actual width whichever is the lesser.

3.2.19.6.2 In the case of transverse members the dispersion width, L_d , considered to carry the track load shall be as follows:

a) transverse troughing:

- 1) cross sleeper track, sleepers in troughs $L_d = 1.5\text{m}$;
- 2) cross sleeper track, up to 150mm depth of ballast under sleeper $L_d = 1.8\text{m}$;
- 3) cross sleeper track, more than 150mm depth of ballast under sleeper $L_d = 2.4\text{m}$;
- 4) longitudinal timber, up to 150mm deep directly on transverse troughing $L_d = 1.5\text{m}$;
- 5) longitudinal timber, over 150mm deep directly on transverse troughing $L_d = 1.8\text{m}$;

b) transverse reinforced concrete slabs, filler joist decks, steel plate decks:

- 1) cross sleeper track, up to 150mm depth of ballast under sleeper and top of the decking $L_d = 1.5\text{m}$;
- 2) cross sleeper track, more than 150mm depth of ballast under sleeper and top of the decking $L_d = 1.8\text{m}$.

3.2.22 Masonry arches

Modified 'Pippard' method

3.2.22.1 MEXE developed a simple experimental method for assessing the capacity of masonry arches for carrying military traffic. It was based on theoretical studies carried out by Professor Pippard supported by observations of the behaviour of arches under actual live loads. The method, which took account of the arch barrel and its geometric properties, was modified by the DoT to suit normal civilian highway traffic. This MEXE/DoT approach could not be applied to railway underbridges, but A.J.S. Pippard's 'The Approximation of Safe Loads on Masonry Bridges' has been used as a basis with condition factors taken from the MEXE/DoT method. Details of the method are given below and it shall be used before more complex methods of assessment are used.

Scope

3.2.22.2 This section deals with the assessment of the strength of the arch barrel only. The strength of the bridge may be affected by for example the strength of the spandrel walls, wing walls and foundations and shall be dealt with under section 14 of this standard. This method shall not be used where the arch is flat (i.e. span/rise ratio greater than 8) or appreciably deformed; or where the span exceeds 20m.

Theory

3.2.22.3 The long term strength of a brick or concrete arch is almost impossible to calculate accurately and recourse has, therefore, been made to an empirical formula based on arch dimensions. The arch is first assumed to be parabolic in

shape with span/rise ratio of 4; soundly built in good quality brickwork/concrete; with well pointed joints; to be free from cracks, and to have adequate abutments. For such an idealised arch, a provisional assessment shall be obtained from graphs in attachment 3. This provisional assessment shall then be modified by factors which allow for the way in which the actual arch differs from the ideal.

3.2.22.4 Survey of arch

3.2.22.4.1 The arch shall be inspected in accordance with section 8 of this standard and the following dimensions measured as shown in figure 4:

- the span l : (in the case of skew spans, measure l parallel to the principal axis of the arch);
- the rise of the arch barrel at the crown r_c ;
- the rise of the arch barrel at the quarter points r_q ;
- the thickness of the arch barrel d ;
- the average depth in metres of fill or ballast between the underside of the sleeper and the extrados of the arch barrel at the crown h .

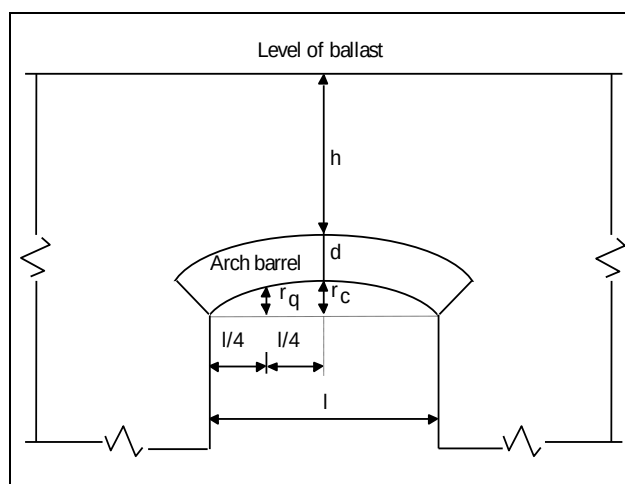


Figure 4 Arch dimensions (metres)

3.2.22.4.2 The following information shall also be required to derive the various modifying factors:

- type of material used for the arch barrel;
- type of construction of the barrel; i.e. are the voussoirs in courses or laid at random?
- condition of materials in the barrel; i.e. is there a lot of spalling and are the voussoirs sound or are they deteriorating due to weathering?
- deformation of the arch barrel from its original shape;
- positions of dropped voussoirs and the amount of drop;

- f) width, length, number and position of cracks;
- g) width of mortar joints;
- h) depth of mortar missing from joints;
- i) condition of joint mortar.

3.2.22.4.3 Note should be taken of any evidence of separation of the arch rings, particularly with regard to any additional rings which have been constructed in later years, and due account shall be taken in the value assumed for the arch barrel thickness. Note shall be taken of any evidence of repairs to the arch rings and due allowance made in the assessment.

3.2.22.5 Provisional assessment

3.2.22.5.1 The provisional axle loading capacity (CP) for single or bogie axles is obtained by reference to graphs in attachment 7.6 – 7.10 which enable CP values to be read for arches of varying spans, ring thickness, and thickness of fill or ballast.

3.2.22.5.2 All arches on the London Underground systems were built in the days of Imperial measurements, and the record drawings are in Imperial units. Graphs in attachment 7.6 – 7.10 are therefore drawn in Imperial units to give a CP in tons. This value shall be multiplied by 1.016 to give a value in tonnes.)

3.2.22.5.3 The CP capacity obtained shall then be modified by the modifying factors in 3.2.22.6

3.2.22.6 Modifying factors

3.2.22.6.1 Span/rise factor (F_{sr})

Flat arches are not so strong under a given loading as those of a steeper profile, and the provisional assessment shall, therefore, be adjusted. A span (L)/rise (r_c) 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 Figure 5 which gives the appropriate span/rise factor, F_{sr} , for the different ratios.

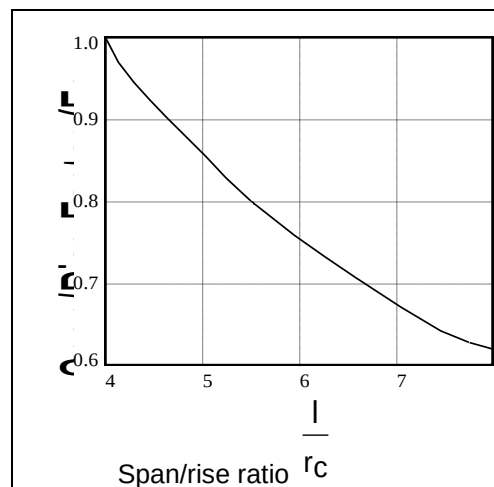


Figure 5 Span/rise factor

3.2.22.6.2 Profile factor (Fp)

There is evidence that elliptical arches are not so strong as segmental and parabolic arches of similar span/rise ratio and barrel thickness. The ideal profile has been taken to be parabolic and for this shape the rise at the quarter points, $r_q = 0.75 r_c$, where r_c is the rise at the crown.

3.2.22.6.3 The profile factor F_p for ratios of r_q/r_c less than or equal to 0.75 shall be taken to be 1, and for ratios greater than 0.75 shall be calculated from the expression:

$$F_p = 2.3 \frac{(r_c - r_q)^{0.6}}{r_c}$$

For convenience this is plotted in figure 6

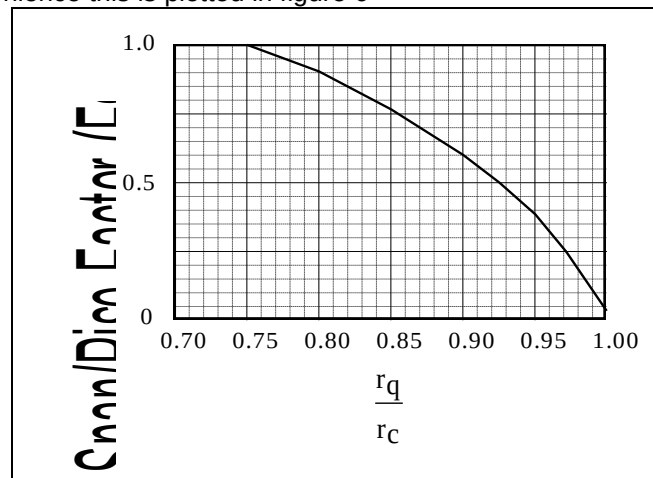


Figure 6 Profile factor

3.2.22.6.4 Condition factor

3.2.22.6.4.1 Material factor (F_m) Arches and viaducts on the LUL system are constructed of either brick or concrete and the material factor F_m shall be one of the following:

- a) building bricks 1.0;
- b) engineering bricks and concrete 1.2.

3.2.22.6.4.2 Joint factor (F_j) The strength and stability of the arch barrel depend to a large extent on the size and condition of the joints. Lime mortar was commonly used in old bridges and, although it is softer than cement mortar and has lower strength, this is compensated for by better joint filling properties and good distributing power under load. The F_j is obtained from the following formula:

Joint factor = width factor x depth factor x mortar factor

$$F_j = F_w \cdot F_d \cdot F_m$$

3.2.22.6.4.3 Appropriate values for F_w and F_m shall be obtained from tables 8 and 9 respectively. The depth factor F_d may be taken as 1.0 for pointed joints in good condition. In the case of insufficiently filled joints, it is recommended that if the depth of missing mortar can be estimated with reasonable accuracy, the thickness of the arch barrel shall be reduced by this amount. When this is not appropriate, the F_d may be taken from table 9.

Width of joint	Width factor (Fw)
Joints with widths up to 6mm	1.0
Joints with widths between 6mm and 12.5mm	0.9
Joints with widths over 12.5mm	0.8

Table 7 Width Factor

Condition of joint	Mortar factor (Fmo)
Mortar in good condition	1.0
Loose or friable mortar	0.9

Table 8 Mortar Factor

Construction of joint	Depth factor (Fd)
Joints in good condition	1.0
Unpointed joints, pointing in poor condition And joints with up to 12.5mm from the edge Insufficiently filled.	0.9*
Joints with from 12.5mm to one tenth of the thickness of the barrel insufficiently filled.	0.8*
Joints insufficiently filled for more than one-tenth of the thickness of the barrel.	At the engineer's discretion
*Interpolation between these values is permitted, depending upon the extent and position of the joint deficiency.	

Table 9 Depth Factor

3.2.22.6.5 **Crack factor (Fcr)** The age of cracks is of great importance. Old cracks, which may have occurred soon after the bridge was built, and are no longer operating can be ignored. Recent cracks, on the other hand, usually show clean faces, with possibly small and loose fragments of masonry. Although cracks appear as shear of the bricks or masonry, they normally follow an irregular line through the mortar. For this reason, care must be taken to ensure that they are cracks and not deficiencies of the pointing material.

3.2.22.6.5.1 Cracks in the abutment can generally be ignored in themselves unless they are new, but if they are caused by subsidence, they may have affected the arch ring, as shown in 3.2.22.6.5.2.

3.2.22.6.5.2 If heavier axle load traffic starts to pass over the bridge after the assessment, any substantial or new abutment cracks shall be re-examined at six monthly intervals to ensure that the cracks are not increasing and thereby affecting the arch ring.

3.2.22.6.5.3 Similarly, any old cracks (mentioned above) shall be re-examined at six monthly intervals after heavier axle loads traffic starts to pass over the bridge to ensure that they are not widening or extending.

3.2.22.6.5.4 The following is a list of the more important types of crack:

a) longitudinal cracks outside the centre third of the arch, caused by lateral spread

of the fill, producing an outward force on the spandrel walls, pushing the outer portion of the ring away from the centre portion;

b) longitudinal cracks within the centre third of the bridge due to varying amounts of subsidence in different places along the length of the abutment. These are dangerous if large, because they tend to indicate secondary breaking up;

c) transverse cracks, usually found near the quarter points, due to permanent deformations of the arch, may be caused by partial collapse of the arch or movement at abutments;

d) diagonal cracks, 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 sides of the abutment and indicate that the bridge could be in a dangerous state;

e) In the case of diagonal cracks meeting or crossing, there is a possibility that a portion at the joint could punch out, as shown in figure 7, and therefore action shall be taken as soon as possible to prevent this happening.

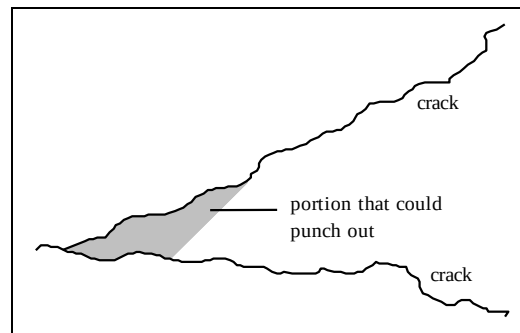


Figure 7 Diagonal cracks

f) cracks between the arch ring and the spandrel or parapet walls. These are due to:

1) spread of the fill so that the parapet wall is pushed out relative to the arch ring, or increased depth of fill or live loading causing the parapet wall to be pushed out relative to the arch ring;

2) movement of a flexible ring away from a stiff fill, so that the two act independently. This type of defect frequently produces cracks in the spandrel wall near the quarter points.

3) The crack factors that shall be applied are given in table 10.

Type of cracks	Factor (Fcr)
i) No cracks	1.0
ii) Longitudinal cracks outside the centre third of the arch, less than one tenth of the span in length.	0.9
iii) Longitudinal cracks outside the centre third of the arch, longer than one tenth of the span in length.	0.9
iv) Longitudinal cracks within the centre	0.9

third of the arch, less than one tenth of the span in length.	
v) Longitudinal cracks within the centre third of the arch, longer than one tenth of the span in length.	0.85
vi) Lateral and diagonal cracks	0.37 – 0.7

Table 10 Crack Factor

3.2.22.6.5 Span factor (F_{sp})

In the case of multi-span arches, each span is to be considered separately and the factors given in table 11 shall be applied.

Span type	Factor F _{sp})
1) Arch supported on one abutment and one pier	0.9
2) Arch supported on two piers	0.8
3) Arch supported on two massive type piers	1.00
4) Arch supported on one abutment and one massive type pier	1.00

Table 11 Span Factor

3.2.22.6.6 Deformation factor (f_d)

This may be due to partial failure of the arch ring, or movement of the abutments, and shall not be mistaken for irregularity due to faulty centres being used during construction. If the deformation results in a flat section, the final axle capacity of the bridge shall be reduced in the proportion of rise at centre of the flattened portion to the original rise at this point. If part of the arch is sagging, the bridge shall not be assessed but remedial action must be taken to strengthen the bridge as a matter of urgency:

$f_d = \text{Rise at centre of flattened portion} / \text{rise at the same point}$

3.2.22.7 Application

Modified axle load

3.2.22.7.1 The span/rise, profile, material, joints, crack, span and deformation factors shall be applied together with the provisional axle loading obtained as directed in 3.2.22.5 in order to determine the modified axle load which represents the allowable loading on the arch from any number of axles.

$$\text{Modified Axle Load} = F_{sr} \cdot F_p \cdot F_m \cdot F_j \cdot F_{cr} \cdot F_{sp} \cdot F_d \cdot C_P$$

3.2.22.7.2 It shall be noted that these allowable axle loads may not represent the strength of the bridge as a whole. This may be affected for example by the strength of the spandrel walls, wing walls and foundations. Should the strength of any of these items be assessed as being lower than the barrel strength, then the lowest value shall be taken as the strength of the bridge as a whole.

3.2.23. Strength of members

3.2.23.1 General

The strength of members shall be assessed in accordance with the relevant requirements of this section. Several modes of failure may need to be considered.

3.2.23.2 Steel

The strength of steel members shall be assessed in accordance with BS 5400: Part 3 amended as described in 3.2.23.3.

3.2.23.3 Main girders restrained by U Frames

Effective length

3.2.23.3.1 In calculating I_1 the inertia of the U frame vertical member, the shape limitations of outstands as specified in BS 5400: Part 3 Clause 9.3.4 shall be ignored (but see clause 13.3.4d of BS5400:Part 3).

3.2.23.3.2 In calculating δ , the U-frame flexibility, the flexibility of the connections shall be determined from attachments 7.11 – 7.21.

3.2.23.3.3 For bridge arrangements other than simple symmetrical U-frames δ shall be calculated from first principles.

3.2.23.3.4 For skew bridges with L-frames restraining the skew ends δ shall be calculated at various cross-sections along the bridge and the maximum value of δ so obtained shall be used to calculate the effective length.

3.2.23.3.5 Slenderness (BS 5400: Part 3 Clause 9.7)

In calculating the slenderness parameter λ_{LT} the value of η shall be taken as 1.0.

3.2.23.3.5.1 The flange area p_f shall be applied if the area of the flanges varies within a distance of $0.7l_e$ on either side of the location for which the moment capacity is being determined.

3.2.23.3.5.2 For the minimum section of a plated girder, the flange area ratio shall be taken as 1.0.

3.2.23.3.6 Restraints to compression flanges (BS 5400: Part 3 Clause 9.12)

Intermediate U-frame forces (BS 5400: Part 3 Clause 9.12.2.3): for intermediate U-frames F_c shall be calculated due to the effects of live load only, unless the skew exceeds 20° and shall include the effects of joint flexibility and flange bending such that:

$$F_c = \frac{\phi d_2}{\delta + \left(\frac{l_u^3}{15 E_c} \right)}$$

Where d_2 is the distance from the centroid of the compression flange to the centroid axis of the deck;

l_u is the distance between U frames;

I_c is the second moment of area of the compression flange about its centroidal axis parallel to the web of the beam at the point of maximum bending moment;

δ is as defined in BS 5400: Part 3 Clause 9.6.5;

ϕ is the maximum difference in end rotation between adjacent U-frame cross girders due to live loading (Live loading for intermediate stiffeners and Live and Dead loadings for bearing stiffeners), assuming the cross-girders are simply supported.

3.2.23.3.6.1 The following is an alternative method for determining F_c forces.

a) For ballasted decks:

Where cross girders are spaced at 1.8m centres or less the equivalent axle load shall be reduced in the ratio: $F = \text{cross girder centres in metres} / 1.8$, e.g. if the cross girder centres are at 1.2m, the cross girder immediately under the axle shall carry $2/3$ of the axle load and the adjacent cross girders on either side shall carry $1/6$ of the axle load.

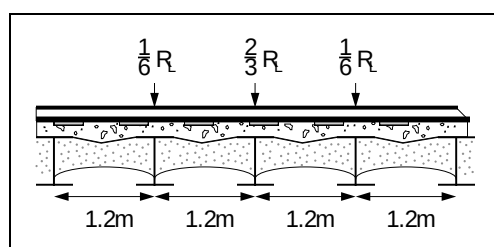


Figure 8 Distribution – ballasted decks

Where the spacing of cross girders is greater than 1.8 metres no distribution shall be allowed.

b) For unballasted decks (longitudinal timbers).

Where the depth of the longitudinal timber is greater than or equal to 225mm, the distribution shall be as for ballasted decks above provided the cross girder spacing are spaced at 1.8 metres or less.

Where the longitudinal timbers are less than 225mm deep (but not less than 150mm depth) and the cross girders are spaced at 1.5 metres or less the equivalent axle load shall be reduced in the ratio: $F = \text{cross girder centres} / 1.5$; e.g. if the cross girders are at 1.2m and the longitudinal timbers are 150mm deep, the cross girder immediately under the axle shall carry $4/5$ of the axle load and the adjacent cross girders on either side shall carry $1/10$ of the axle load.

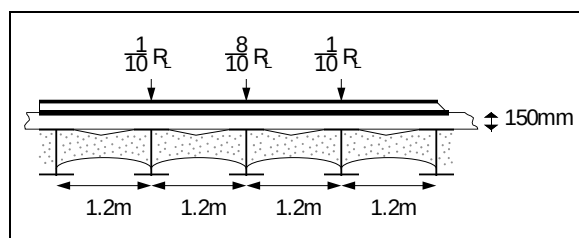


Figure 9 Distribution – unballasted decks

Where cross girders are spaced at more than 1.5 metres and with longitudinal timbers less than 225mm deep no distribution will be allowed.

For a rigorous assessment ϕ shall be determined from the influence lines for end rotation of the cross-girders which shall include the effects of skew where present.