VOLUME 2 HIGHWAY STRUCTURES: DESIGN (SUB-STRUCTURES AND SPECIAL STRUCTURES) MATERIALS

SECTION 2 SPECIAL STRUCTURES

PART 14

BD 91/04

UNREINFORCED MASONRY ARCH BRIDGES

SUMMARY

This document sets the Standard requirements for, and gives advice on, the design of unreinforced masonry arch bridges.

INSTRUCTIONS FOR USE

This is a new document to be incorporated into the Manual.

1. Remove Contents pages for Volume 2.

2. Insert new Contents pages for Volume 2, dated November 2004.

3. Insert BD 91/04 into Volume 2, Section 2, Part 14.

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THE DEPARTMENT FOR REGIONAL DEVELOPMENT
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Unreinforced Masonry Arch Bridges

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REGISTRATION OF AMENDMENTS

Amend No	Page No	Signature & Date of incorporation of amendments	Amend No	Page No	Signature & Date of incorporation of amendments

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PART 14

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1. INTRODUCTION

1.1 General

1.1.1 Experience has shown that arch bridges are very durable structures requiring little maintenance in comparison to other bridge forms. BD 57 (DMRB 1.3.7) says that their use should be considered. However, there has not previously been a standard for the design of new unreinforced arch bridges. The objective of this Standard is to encourage a renaissance in arch building using unreinforced masonry materials.

1.1.2 Research into the behaviour of arch bridges has been undertaken by a number of organisations including Transport Research Laboratory, British Rail Research and a number of universities. In this Standard, consideration has been given to the results of most of this work.

1.1.3 Maintenance requirements have significant effects on whole life costs. The financial benefits arising from reduced maintenance requirements for unreinforced arch bridges should be considered when comparing the whole life costs of arch bridges with other types of bridges.

1.1.4 This Standard states the design requirements for arch bridges. It will complement the new additions for unreinforced masonry arch bridges, referred to hereafter as 'arch bridges', in the Specification for Highway Works (MCHW1) which is hereafter referred to as the 'Specification'. Background information for this Standard is given in Annex A.

1.2 Scope

1.2.1 This Standard applies to arch bridges consisting of single or multiple spans, right or skewed with a span/rise ratio of between 2 and 10 and spans not exceeding 40m.

1.2.2 Open spandrel arch bridges and arch bridges carrying railway loading are excluded from this Standard.

1.3 Symbols

b	width of the arch ring under consideration
e	eccentricity of the centre of compression in the arch ring
f_k	characteristic compressive strength of masonry
h	overall thickness of the arch ring
E_d	design load effects
F_d	design load
F_k	nominal load
P	axial force in arch ring
R_k	design resistance of structural member
S	length loaded with SV vehicle
V	shear force
γ_f	partial factor for load
$\gamma_{G,sup}$	partial factor for permanent load in calculating upper design value
$\gamma_{G,inf}$	partial factor for permanent load in calculating lower design value
γ_{Rd}	partial factor for material
ψ	load combination factor

1.4 Equivalence

1.4.1 The construction of arch bridges will normally be carried out under contracts incorporating the Specification. In such cases, products conforming to equivalent standards or technical specifications of other states of the European Economic Area, and tests undertaken in other states of the European Economic Area, will be acceptable in accordance with the terms of Clauses 104 and 105 of the Specification.

Any contract not containing these clause shall contain suitable clauses of mutual recognition having the same effect, regarding which advice should be sought.

1.5 Implementation

1.5.1 This Standard shall be used forthwith on all schemes for the construction and improvement of trunk roads, including motorways, currently being prepared, provided that in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay progress. Design Organisations shall confirm its application to particular schemes with the Overseeing Organisation. For use in Northern Ireland, this Standard will be applicable to those roads designated by the Overseeing Organisation.

1.6 Definitions

1.6.1 The following definitions apply to common terms used in this Standard. Definitions of other specific terms are given as they arise within the various clauses or in the references quoted.

Abutment is the part of a bridge which provides resistance to horizontal and vertical forces from an arch ring.

Arch ring is a curved course of masonry, or series of masonry courses, which supports loads principally in compression.

Extrados is the convex surface of an arch ring.

Fill is the material placed above the extrados, which may include a pavement sub-base.

Foundation is that part of the structure in direct contact with and transmitting loads to the ground.

Intrados is the concave surface of an arch ring.

Masonry is an assemblage of structural units usually laid in-situ in which the structural units, usually clay bricks, concrete blocks or stones, are bonded and solidly put together with mortar.

Parapet base slab is the foundation which supports the bridge parapet.

Pavement is the bound material forming footpath/verge or carriageway and includes surfacing and roadbase as appropriate, but excludes sub-base.

Pier is an intermediate support between adjoining arch spans.

Rise is the vertical height from the springing level to the crown of the intrados.

Skewback is the surface of an inclined springing.

Span is the clear distance between the faces of the abutments or piers.

Spandrel wall is the wall carried on the arch extrados, which retains the fill.

Springing is the plane from which an arch ring springs.

Unreinforced masonry is masonry which does not include steel or other reinforcement which is considered in the determination of its strength.

String Course is a moulded course that projects from a wall.

Voussoir is a wedge shaped masonry unit in an arch.

Wing wall is a wall at the abutment which extends beyond the spandrel walls to retain the earth behind the abutment.

1.6.2 Figure 1.1 illustrates various features of an arch bridge.

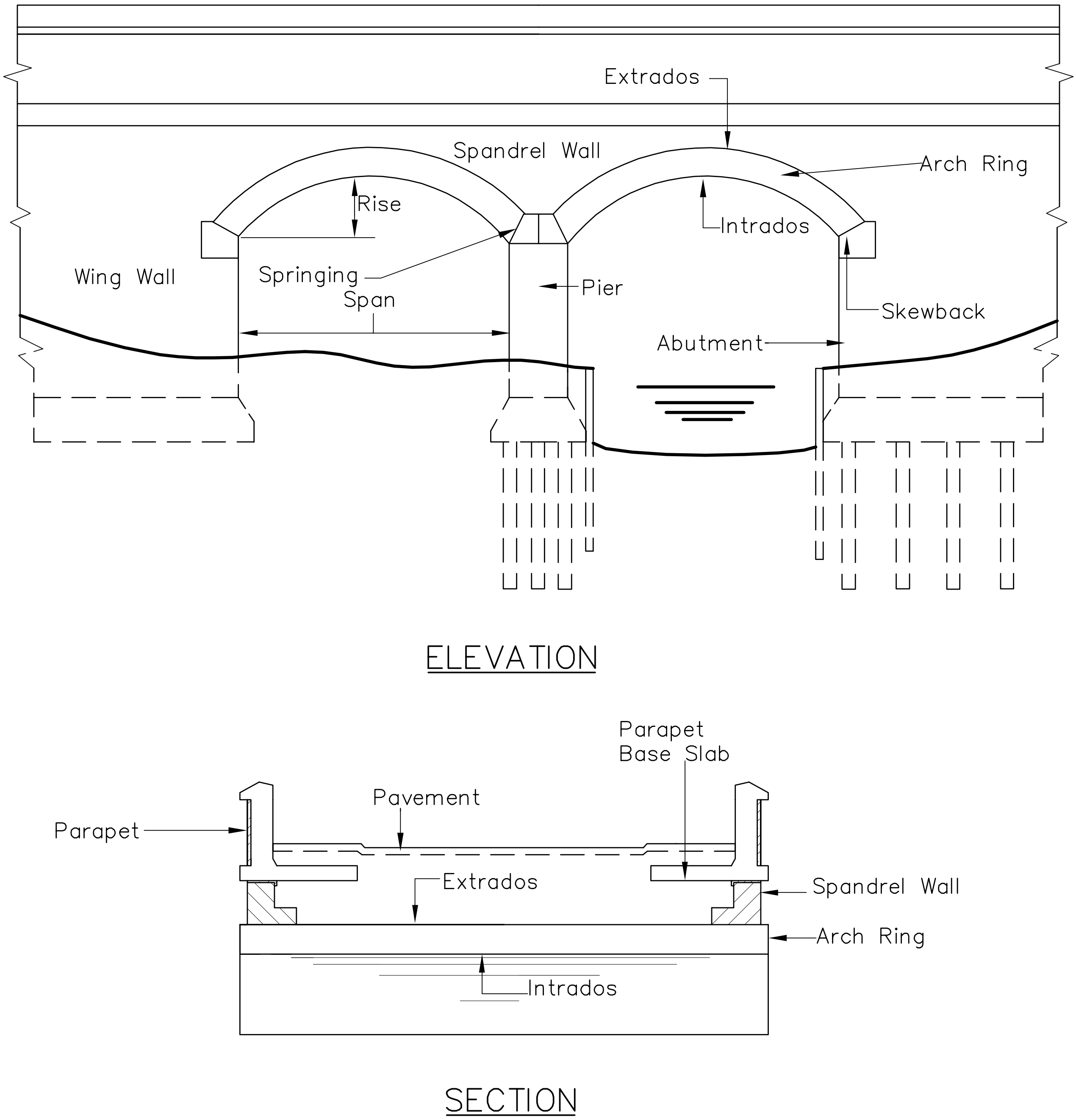


Figure 1.1 Features of an arch bridge

1.7 Mandatory Requirements

1.7.1 Sections of this document which form mandatory requirements of the Overseeing Organisation are highlighted by being contained within boxes. The remainder of the document contains advice and enlargement which is commended to designers for their consideration.

2. DESIGN PRINCIPLES AND OBJECTIVES

2.1 General

2.1.1 This Standard adopts a limit state partial factor approach, as described in Section 6 of BS EN 1990, in which a structure shall be shown to be safe by the application of partial safety factors to loads (γ_f) and to material strengths (γ_{Rd}).

2.1.2. The design life shall be 120 years.

2.1.3 Each structure and each part of a structure is required to fulfil fundamental requirements of stability, strength, stiffness and serviceability during construction and throughout its design life.

2.1.4 Whenever a structure, or part of a structure, fails to satisfy one of the fundamental requirements it is said to have reached a “limit state”.

2.2 Limit States

2.2.1 The structure and associated earthworks, including the fill and foundations shall be designed to perform satisfactorily at both the ultimate and serviceability limit states. These two limit states, which are to be considered in the design, are described in 2.2.2 and 2.2.3.

2.2.2 The Ultimate Limit State (ULS) is the condition at which a collapse mechanism forms in the structure or when movements of any part of the structure lead to severe structural damage in other parts of the structure or services.

Requirements for the ultimate limit state are given in Chapter 4.

2.2.3 The Serviceability Limit State (SLS) is the condition beyond which there is a loss of utility due to any of the following:

(i) Deformation of the structure causing a loss of utility or adversely affecting its appearance to a point where public concern may be expected.

(ii) Cracks become of such magnitude as to lead to a reduction in structural integrity.

(iii) Repeated loading reduces the ultimate capacity of the structure, (fatigue).

Requirements for the serviceability limit state are given in Chapter 4.

2.3 Nominal Loads

2.3.1 The loads to be considered in determining the load effects, E_d, on the structure are specified in Chapter 3 and are described as nominal loads, F_k.

2.4 Design Loads

2.4.1 The design loads, F_d, are determined from the nominal loads, F_k, according to the relationship:

$$F_d = \gamma_f F_k$$

where γ_f is a partial safety factor that takes account of the possibility of an unfavourable deviation of the loads from their nominal. Where more than one transient load type is applied simultaneously a combination factor ψ is applied to take account of the reduced probability that various loadings acting together will attain their nominal values simultaneously.

2.4.2 Values of γ_f are given in Chapter 3. Values of ψ, where required, shall be agreed with the Overseeing Organisation.

2.5 Design Load Effects

2.5.1 The design load effects, E_d, are obtained from the design loads, $F_{d,}$ by the relationship:

$$E_d = (\text{ effects of } F_d\ )$$
$$E_d = (\text{ effects of } \gamma_f F_k\ )$$

2.6 Design Resistance

2.6.1 The design resistance of structural elements, R_d, is defined as:

R_d = function of design strength of material considered

$= R_k / \gamma_{Rd}$

where R_k is the characteristic (or nominal) strength of the material.

γ_{Rd} is a partial safety factor to cover possible reductions in the strength of the materials in the structure as a whole compared with the value deduced from the control test specimens, and to cover possible weaknesses of the structure arising from any other cause including manufacturing tolerances.

2.6.2 Values of γ_{Rd} are given in Chapter 4.

2.6.3 The design resistance of the sub-soil and fill shall be in accordance with BD 74 (DMRB 2.1.8). The arch ring is sensitive to the effects of foundation movements. The effects of estimated displacements and rotations over a period of 120 years shall be considered.

2.7 Compliance

2.7.1 All elements of the structure, and the structure as a whole, shall comply with the requirements for the ultimate limit state and the serviceability limit state.

2.7.2 The following relationship shall be satisfied for each limit state:

$$\mathbf{R_d \geq E_d}$$

2.7.3 Analysis shall be undertaken to ascertain load effects for each of the most severe conditions appropriate to the part under consideration. The method of analysis shall be capable of predicting all significant load effects. Analysis of the arch ring shall make due allowances, where appropriate, for elastic shortening, loss of stiffness due to cracking, creep, shrinkage and other predictable deformations as these deformations may significantly modify load effects. An indication of when this is likely is given in 4.1.3.

3. ACTIONS

3.1 General

3.1.1 As this Standard follows the BS EN 1990 and BS EN 1991 approach to actions, it means that there is no separate load factor γ_{f3} as given in BD 37 (DMRB 1.3.14). The values of the individual load factors have to be increased to compensate. The loading Standard BD 37 (DMRB 1.3.14) is therefore not applicable, although where loading is not given in this Standard and the relevant parts of BS EN 1990 and BS EN 1991 are not yet available, it may be used to derive the loading with the agreement of the Overseeing Organisation.

3.1.2 Permanent actions shall be determined in accordance with BS EN 1991-1-1, as modified by 3.2.

3.1.3 Where bridges are likely to be subject to scouring or any other hydraulic action, designs should take into consideration the recommendations of BA 59 (DMRB 1.3.6).

3.2 Permanent Actions

3.2.1 For design loads, the partial factor for load, γ_G, to be applied to the nominal permanent load shall be taken as follows:

	ULS (Adverse, $\gamma_{G,sup}$)	ULS (Relieving, $\gamma_{G,inf}$)	SLS
Masonry	1.35	0.95	1.0
Foamed concrete	1.35	0.95	1.0
Other Fill	1.2	0.95	1.0
Surfacing	1.2	0.95	1.0

3.3 Thermal Action

3.3.1 Generally, thermal action is not critical to stresses except in very flat arches and may be discounted, subject to agreement with the Overseeing Organisation, except as noted in 4.1.3.

3.3.2 Where thermal action has to be considered, it will be considered in accordance with BS EN 1991-1-5.

Values for load combination factors, ψ, shall be agreed with the Overseeing Organisation.

3.3.3 For the determination of thermal effects, masonry arch bridges shall be classified as type 3 structures.

3.3.4 The minimum uniform (effective) bridge temperatures given by BS EN 1991-1-5 may be increased by +1°C for each 100mm of cover above the crown. The maximum uniform (effective) bridge temperatures given by BS EN 1995-1-5 may be reduced by 2°C for each 100mm of cover above the crown. These adjustments should not be applied beyond a limiting differential between maximum and minimum uniform (effective) bridge temperatures of 15°C.

3.3.5 Changes in uniform (effective) bridge temperature may be ignored when the total depth of pavement and fill above the extrados is 1.5 metres or greater.

3.3.6 For the purpose of establishing temperature differences, the depth of fill shall be included in "Depth of slab (h)".

3.3.7 Heating (positive) temperature differences may be ignored when the total depth of pavement and fill above the extrados exceeds 500mm.

3.3.8 Cooling (negative) temperature differences on the extrados may be ignored when the total depth of pavement and fill above the extrados exceeds 500mm.

3.3.9 If thermal expansion is likely to be a critical factor in the design, the coefficient of thermal expansion of the masonry to be used should be established by testing. Otherwise the coefficient of thermal expansion for masonry may be taken as:

10×10^{-6}/°C for masonry with concrete units,
6×10^{-6}/°C for masonry with clay units,
5 to 13×10^{-6}/°C for masonry with reconstituted stone units.

For natural stone masonry the coefficient of thermal expansion should be determined for the actual rock type to be used in the construction.

3.4 Wind Action

3.4.1 Generally wind action is not critical, but should be considered at the designer's discretion. The wind loads may be based on BD 37 (DMRB 1.3.14) or when available BS EN 1991-1-4.

3.5 Traffic Loads

3.5.1 Vehicles to be Considered

3.5.1.1 Neither BD 37 (DMRB 1.3.14) HA UDL and HA KEL nor BS EN 1991-2 LM1 and LM2 loadings satisfactorily model the effect of Authorised Weight Regulation Vehicles on masonry arches. The load on all such structures carrying roads shall be determined directly by considering individual vehicles, or combinations of vehicles. The vehicles to be considered, which are based on Authorised Weight regulations, are given in 3.5.4.1.

3.5.1.2 Motorway and Trunk Road bridges shall also be checked for the SV vehicles given in Table 3.1.

The requirements for Principal Roads and Other Public Roads may be specified by the Overseeing Organisation. Recommendations are given in Table 3.1. Accommodation bridges will not normally be checked for these vehicles.

3.5.1.3 The following loadings do not necessarily cover the effects of Special Order Vehicles. The design loadings for bridges required to carry these vehicles shall be agreed with the Overseeing Organisation.

Class of Road Carried by Structure	**SV Vehicles to be Considered**
Motorway and Trunk Roads (or principal road extensions of trunk routes)	SV 100, SV 196, SV TT
Principal Roads (recommended minimum)	SV100
Other Public Roads (recommended minimum)	SV 80

Table 3.1 SV Vehicles to be Considered

3.5.2 Lanes

3.5.2.1 Carriageways which are 5.4m or wider but less than 6m wide shall be divided into two equal notional lanes. All other carriageways shall be divided into the maximum integer number of 3m notional lanes and a remaining area.

3.5.2.2 The full effects of loading from AW vehicles or single axles in two adjacent lanes only shall be considered. For vehicles in the 3rd lane a factor of 0.5 shall be applied and for vehicles in all other lanes a factor of 0.4 shall be applied.

3.5.2.3 The position of the notional lanes and remaining area shall be arranged so as to give the most adverse overall effect for the particular verification being considered and the lane loadings shall be interchangeable to give the worst effect.

3.5.2.4 The remaining area shall be loaded with a UDL of 5.0kN/m^2 except where this has a relieving effect.

3.5.3 Dispersal

3.5.3.1 Where the analytical model used does not model dispersal effects through the fill and surfacing, traffic loads may be dispersed through the pavement and fill to the extrados at a spread to depth ratio of 1 horizontally to 2 vertically.

3.5.4 Authorised Weight Vehicles

3.5.4.1 The vehicles to be considered are given in Table 3.2. It is necessary to consider all these vehicles to determine the most onerous effects. The axle loads shall be multiplied by the axle impact factor where required by 3.5.4.4 and by a contingency factor of 1.1. The resulting loads shall be considered as nominal loads.

3.5.4.2 The axles shall be assumed to consist of two equal wheel loads at 1.8m track centre to centre. The wheel loads shall be assumed to be uniformly distributed over wheel contact areas which may be assumed to be either 300mm squares or 340mm diameter circles. The minimum transverse distance between the centre of a wheel and that of the wheel of another vehicle shall be 0.7m. The vehicle shall be positioned so that it is wholly within one lane.

3.5.4.3 The vehicles given in Table 3.2 shall be positioned to give the worst overall effect.

3.5.4.4 The following loads shall be applied in each notional lane:

Case 1 Single vehicle with axle impact factor of 1.8 applied to one axle.

Case 2. Convoy of vehicles (jam situation with no axle impact).

For case 2, the minimum distance between vehicles shall be 1m.

3.5.4.5 For structures where the critical load case is due to two or more vehicles spaced at more than 10m, the axle impact factor of 1.8 shall be applied to the critical axle of one vehicle within the lane.

3.5.4.6 In addition, arches shall be separately checked for a single axle consisting of 2 no. 100kN loads with the geometry given in 3.5.4.2.

3.5.4.7 The loads given in 3.5.4.1 to 3.5.4.6 and the remaining area load defined in 3.5.2.4 shall be considered as nominal loads. They shall be multiplied by the load factor γ_f 1.65 to obtain design ultimate load or 1.2 to obtain design service load.

Vehicle Gross Weight (tonnes)	No. of Axles	AXLE WEIGHTS AND SPACING												
		01 (m)	W1 (tonnes)	A1 (m)	W2 (tonnes)	A2 (m)	W3 (tonnes)	A3 (m)	W4 (tonnes)	A4 (m)	W5 (tonnes)	A5 (m)	W6 (tonnes)	02 (m)
32[1]	4	1.0	6.50	1.20	6.50	3.90	11.50	1.30	7.50					1.0
38[2]	4	1.0	6.50	3.00	11.50	5.10	10.00	1.80	10.00					1.0
40[3]	5	1.0	6.00	3.00	11.50	4.20	7.50	1.35	7.50	1.35	7.50			1.0
40[4]	5	1.0	6.00	2.80	11.50	1.30	6.50	5.28	8.00	1.02	8.00			1.0
40[3]	5	1.0	5.00	2.80	10.50	1.30	4.50	4.80	10.00	1.80	10.00			1.0
41[6]	6	1.0	5.00	2.80	10.50	1.30	5.00	4.18	6.83	1.35	6.83	1.35	6.83	1.0
44[7]	6	1.0	6.00	2.80	10.50	1.30	5.00	4.70	7.50	1.35	7.50	1.35	7.50	1.0
44[8]	5	1.0	7.00	2.80	11.50	1.30	7.50	7.60	9.00	1.35	9.00			1.0

Notes

1 4-axle rigid
2 2+2 artic
3 2+3 artic
4 3+2 artic, W2 and W3 can be reversed for worst effect
5 3+2 artic, with 10.5 tonne drive axle, W2 and W3 can be reversed for worst effect
6 3+3 artic, maximum axle weight 10.5 tonnes, W2 and W3 can be reversed for worst effect
7 3+3 artic, maximum axle weight 10.5 tonnes, W2 and W3 can be reversed for worst effect
8 3+2 artic, 40ft ISO container, international intermodal journeys only, W2 and W3 can be reversed for worst effect

Key: 01 and 02 overhang (m)
W1, W2 etc – axle weights (tonnes)
A1, A2 etc – axle spacings (m)

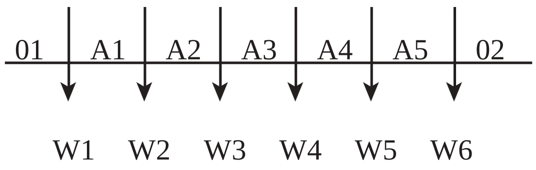
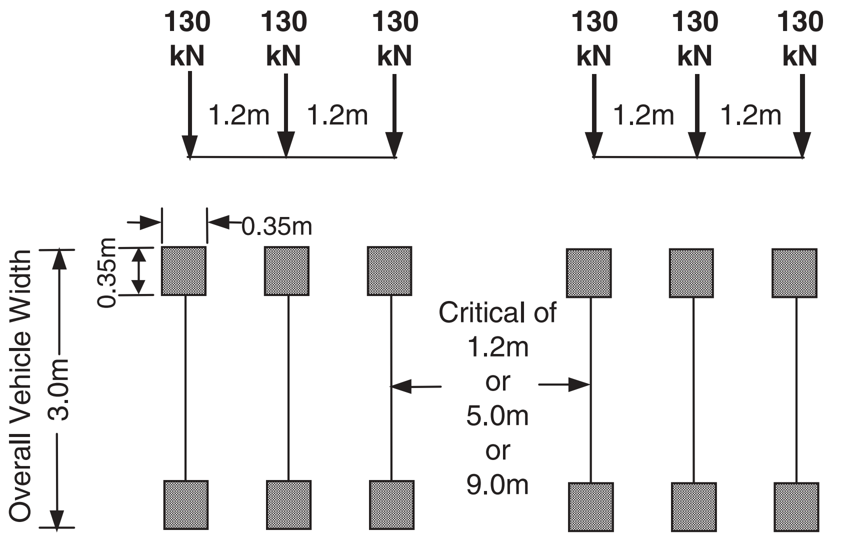


Table 3.2 Vehicles from Authorised Weight Regulations

3.5.5 SV Vehicles

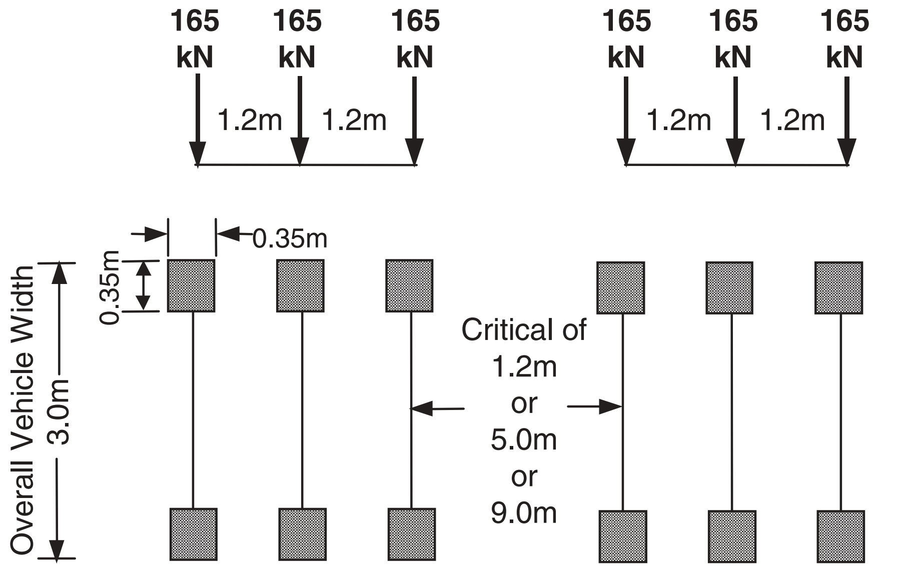
3.5.5.1 Figures 3.1 to 3.5 give the vehicles to be considered in accordance with Table 3.1.

3.5.5.2 Only one SV vehicle shall be applied at any one time and this shall be positioned to give the worst overall effect for the aspect being considered.



Note: Overall vehicle width = overall track

Figure 3.1: SV80 vehicle



Note: Overall vehicle width = overall track

Figure 3.2: SV100 vehicle

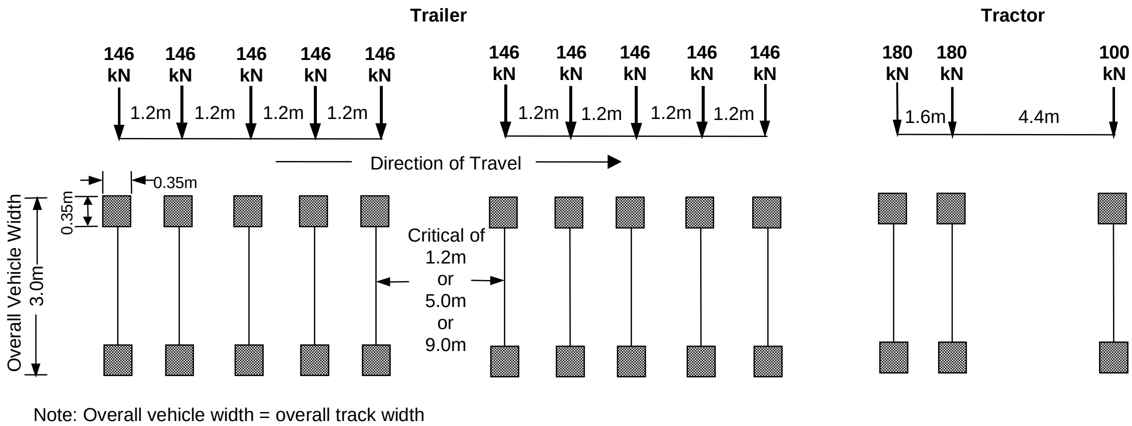


Figure 3.3: SV196 vehicle

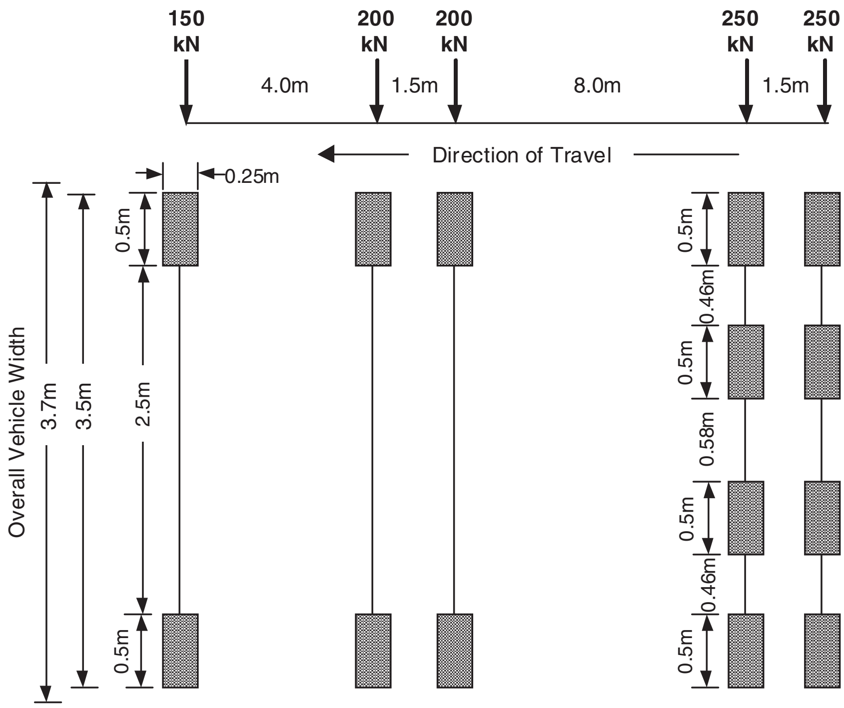


Figure 3.4: SV TT vehicle

3.5.5.3 The wheel loads shall be uniformly distributed over a square or rectangular contact area as shown in Figures 3.1 to 3.4.

3.5.6 Associated AW Vehicles

3.5.6.1 The effects of normal vehicles (those that conform to the AW or C&U Regulations) associated with SV vehicles shall be represented by AW vehicles in accordance with Table 3.2.

3.5.6.2 Remaining area load in accordance with 3.5.2.4 shall be applied to areas which are not covered by lanes and which are not loaded with SV vehicles.

3.5.6.3 Separate assessment for single axle load from AW vehicles associated with an SV vehicle is not required.

3.5.7 Application of SV Vehicles and Associated AW Vehicles

3.5.7.1 SV vehicles and the associated AW vehicles and remaining area load if any shall be considered as nominal loads and shall be multiplied by the load factor γ_f 1.35 to obtain design ultimate and 1.15 to obtain design service load.

3.5.7.2 AW vehicles shall be applied in each notional lane in accordance with 3.5.4.4.

3.5.7.3 Only one SV vehicle shall be considered on any one superstructure.

3.5.7.4 SV vehicles shall be applied in their entirety and shall not be truncated.

3.5.7.5 The SV vehicle shall be placed at any transverse position on the carriageway, either wholly within one notional lane or straddling between two adjacent lanes with its side parallel to the kerb at the most unfavourable position to produce the most severe overall effect.

3.5.7.6 AW loading shall be applied to the notional lanes of the carriageway in accordance with 3.5.4. The axle impact factor shall not be applied to the vehicles in the same lane as the SV vehicle.

3.5.7.7 Where the SV vehicle lies partially within a notional lane and the remaining width of the lane, measured from the side of the SV vehicle to the far edge of the notional lane, is less than 2.5m the associated AW vehicles shall be applied as in the case of the SV vehicle lying fully within a notional lane. Where the remaining width of the lane is greater than or equal to 2.5m, the AW loading in that lane shall remain but the 1.8 impact factor to the critical axle shall not be applied.

3.5.7.8 On the remaining lanes not occupied by the SV vehicle, the associated AW vehicles with appropriate Lane Factors shall be applied in accordance with 3.5.2.2.

3.5.7.9 The lane factors applied to AW lane loadings in accordance with 3.5.2.2 shall be interchangeable for the worst effect.

3.5.7.10 Typical examples of application of SV and AW vehicles are shown in Figures 3.5 and 3.6. Figure 3.5 shows the case of the SV vehicle fitting in one lane and Figure 3.6 shows the case where it straddles lanes.

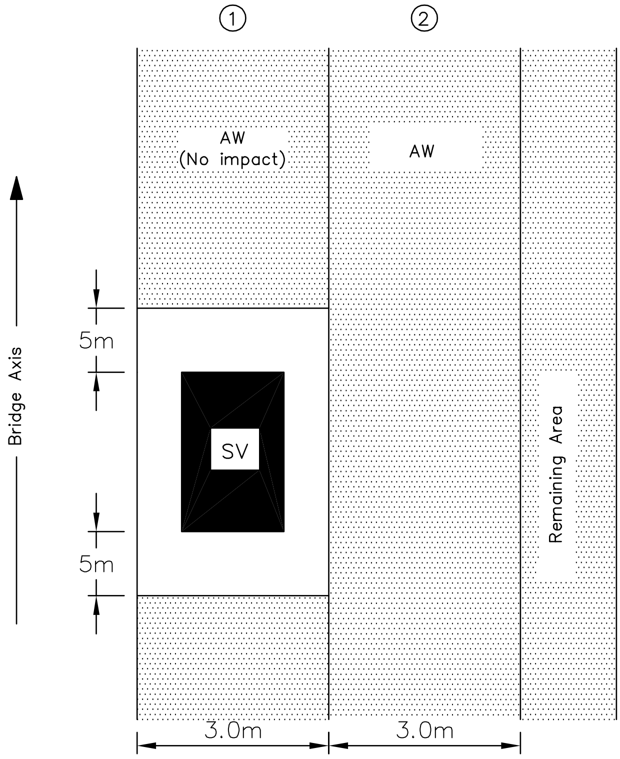


Figure 3.5: Application of SV and AW Vehicle (case where SV vehicle fits in a lane)

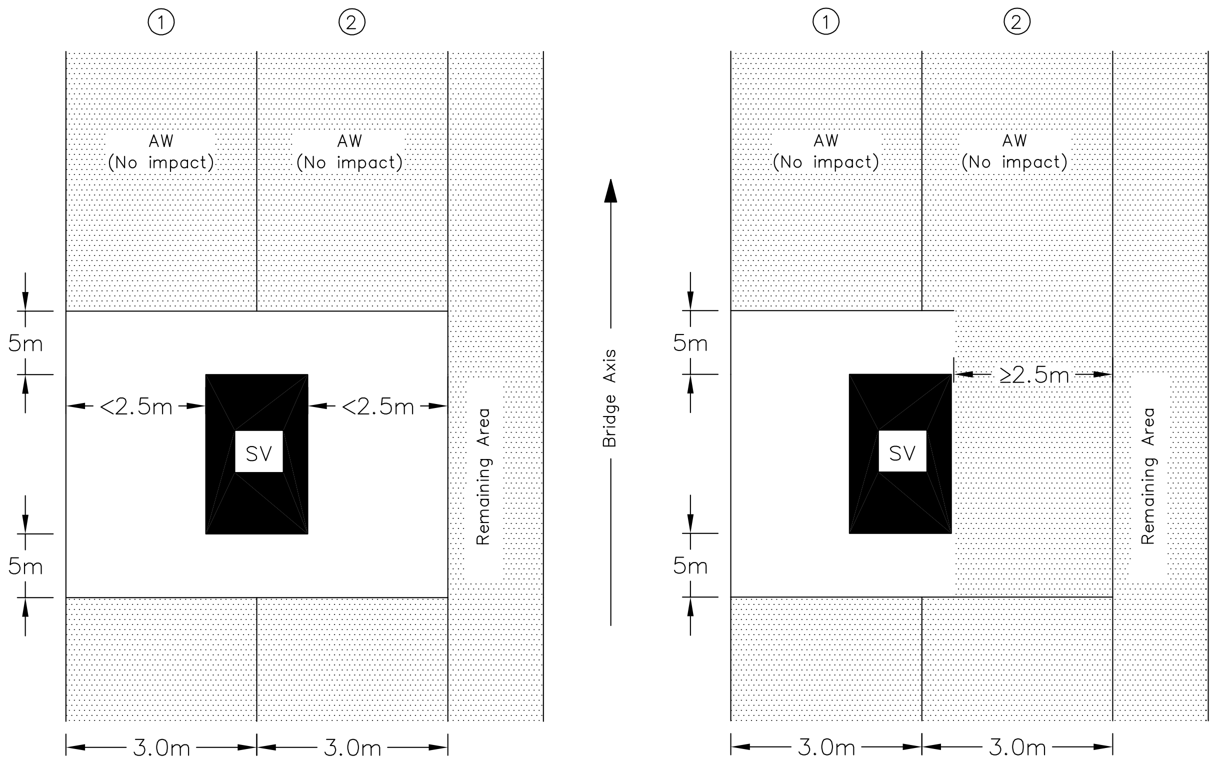


Figure 3.6: Application of SV and AW Vehicles (case where SV vehicle straddles lanes)

3.5.8 Longitudinal Loading

3.5.8.1 Longitudinal loads resulting from traction or braking of vehicles shall be taken as the more severe design load resulting from 3.5.8.2 and 3.5.8.3, applied at the road surface and parallel to it in one notional lane only. However longitudinal loads may be ignored for single span structures.

3.5.8.2 For multispan structures, longitudinal loads associated with AW vehicles shall be taken as 0.6 times the total vertical load in the heaviest loaded lane subject to a maximum of 900kN.

These loadings shall be considered as nominal and shall be multiplied by the load factor g_f 1.35 to obtain the design ultimate load. They need not be considered at SLS.

3.5.8.3 Where longitudinal loading is considered in accordance with 3.5.8.2, the loadings corresponding to SV vehicles shall be taken from Table 3.3 but not need be applied together with AW vehicles. These loadings shall be considered as nominal and shall be multiplied by the load factor g_f 1.35 to obtain design ultimate. They need not be considered at SLS.

Length Loaded (S) (m)	Nominal longitudinal load (kN)		
	SV196	SV100	SV80
S < 1.2	129	118	93
1.2 ≤ S < 1.6	236	236	186
1.6 ≤ S < 2.4	257	236	186
2.4 ≤ S < 3.6	327	327	257
3.6 ≤ S < 4.8	363	363	286
4.8 ≤ S < 6.0	408	408	322
6.0 ≤ S < 7.2	436	436 (for S ≥ 6.0)	343 (for S ≥ 6.0)
7.2 ≤ S < 8.4	445		
8.4 ≤ S < 9.6	472		
9.6 ≤ S < 12.8	490		
12.8 ≤ S < 13.6	500		
13.6 ≤ S < 14.0	504		
14.0 ≤S < 19.6	508		
S ≥ 19.6	535		

Table 3.3: Nominal Longitudinal Load for SV Vehicles

3.5.9 Accidental Vehicle

3.5.9.1 The elements of structure supporting outer verges, footways, central reserves or cycletracks shall be designed to sustain the effects of any one AW vehicle from Table 3.2 applied with the 1.1 contingency factor and with the impact factor of 1.8 applied to the critical axle.

3.5.9.2 No other traffic load, except those loads due to changes in speed or direction of the vehicle traffic, eg longitudinal and skidding loads, shall be applied in combination with it.

3.5.9.3 These loads shall be considered as nominal load and shall be multiplied by the load factor γ_f 1.65 to obtain design ultimate load or 1.2 to obtain design service load.

3.5.10 Footway and Cycletrack Loading

3.5.10.1 Footway or cycletrack loading shall be $5kN/m^2$. This shall be considered as nominal load and shall be multiplied by the load factor γ_f 1.35 to obtain design ultimate load and 1.15 to obtain design service load.

3.5.10.2 Where footway and cycletrack loading is significant, as in footway and cycletrack bridges, it is necessary to consider the length which is loaded to give the worst case. This is **not** normally the full length of the span or bridge and would more typically be one half of a span loaded.

3.5.11 Vehicle collision loads on bridge supports and superstructures

3.5.11.1 The collision loads to be adopted and the safety barrier provisions at bridge supports shall be agreed with the Overseeing Organisation. Generally, the headroom clearance and collision loads shall be in accordance with TD 27 (DMRB 6.1.2) and BD 60 (DMRB 1.3.5) respectively.

4. DESIGN AND RESISTANCES

4.1 General

4.1.1 The effects of permanent actions and all possible combinations of variable actions and accidental actions shall be considered when verifying the adequacy of an arch bridge and its components.

4.1.2 The structure may be analysed using any appropriate analytical model or computer program, subject to the requirements of 4.2 to 4.7.

4.1.3 For bridges with a span/rise ratio greater than 6, the analytical model shall consider the flexibility of the arch ring and supports (abutments and piers) and the effects of creep, shrinkage and temperature.

4.1.4 It is anticipated that bonds where the rings are essentially separate will be avoided as it has been shown[5.4.1] that this can significantly reduce strength. However, in some cases, notably in skew arches, bonding the rings together can be difficult and significantly increase costs. In such cases it would be possible to use separate rings if the analytical approach allowed for this. Another possibility may be to use partially bonded rings, but it would then be necessary to check the interface stresses.

4.2 Materials

4.2.1 Characteristic Strength

The characteristic compressive strength of masonry, f_k may be obtained from BS 5628 Part 1. Pending revision of this British Standard to bring it fully into line with Europen brick standards, this should be used with unit strengths as defined by BS 3921. See Annex A4.

Alternatively, where the use of BS 5628 would require tests, approximate values may be obtained form BD 21 (DMRB 3.4.3).

4.2.2 Elastic Modulus

In the absence of more accurate determination, the short term elastic modulus of masonry (in N/mm^2) may be taken as 900 000f_k.

4.2.3 Creep and Shrinkage

In the absence of more accurate determination, the creep factor (the ratio of creep to short term elastic strain per unit stress) and shrinkage strain may be taken from Table 4.1.

Type of Unit	Creep Factor	Shrinkage
Clay	1.5	0
Stone	1.0	0
Concrete	3.0	500×10^{-6}
Reconstituted Stone	3.0	500×10^{-6}

Table 4.1: Creep and Shrinkage

4.3 Arch Ring - Ultimate Limit State

4.3.1 General

The design of the arch ring under design loads appropriate to this limit state shall ensure that prior collapse does not occur as a result of buckling, instability or rupture of one or more critical sections.

4.3.2 Direct Stresses

4.3.2.1 Tensile strength shall be ignored in the analysis. Where the moment at a section is such as to cause the centre of compression to be outside the middle third, the section shall be assumed to be cracked with a reduced area resisting compressive forces. The maximum compressive stress in the ring shall not be taken as greater than the compressive strength of masonry, f_k, times $0.6/\gamma_{Rd}$ where the material partial factor γ_{Rd} is taken as 1.5.

4.3.2.2 The above may be ensured by ensuring the following relationship is satisfied at all positions in the arch ring.

$$P \leq 0.4\, b\, f_k\, (h - 2e)$$

where P is the compressive force in the arch ring due to ultimate design load effects, E_d

b is the width of the arch ring under consideration

f_k is the compressive strength of masonry

h is the overall thickness of the arch ring

e is the eccentricity of the centre of compression in the arch ring.

The above equation includes an allowance for γ_{Rd}.

4.3.3 Shear

Shear forces on a radial plane through the arch ring shall be checked. The following relationship shall be satisfied at all positions on the arch ring.

$$V \leq 0.4\, P$$

Where P is the compressive force in the arch ring due to ultimate design load effects, E_d

and V is the shear force due to ultimate design load effects, E_d.

4.4 Arch Ring - Serviceability Limit State

4.4.1 Except for bridges required by 4.1.3 to be checked for imposed deformations, analysis for the serviceability limit state is not required if the structure is designed for the ultimate limit state using load factors γ_f of 2.0 for AW vehicles, 1.7 for SV vehicles and AW vehicles associated with SV vehicles.

4.4.2 Where checks are undertaken but more rigorous analysis is considered unnecessary, an arch ring may be designed so that the serviceability design load effects, E_d, satisfy the following conditions:

(1) the eccentricity of the centre of compression, e, does not exceed 0.25h

(2) the compressive stress does not exceed 0.4 f_k

where h is the overall thickness of the arch

and f_k is the characteristic compressive strength of masonry.

4.5 Spandrel Walls, Wing Walls and Abutments

4.5.1 Spandrel walls, wing walls and abutments shall be designed in accordance with Clauses 4.3 to 4.4 and BD 30 (DMRB 2.1.5).

4.5.2 At the ultimate limit state, stability shall be checked against overturning, sliding and bearing (where appropriate), with the application of the following pressures:

(i) foamed concrete fill - hydrostatic pressure of wet concrete;

(ii) class 6N, 6P, 7A or 7B fill - “active” earth pressure.

At both the ultimate and serviceability limit states, the structural design of the wall shall be based on the following pressures:

(i) foamed concrete fill - hydrostatic pressure of wet concrete;

(ii) class 6N, 6P, 7A or 7B fill - “at rest” earth pressure.

4.5.3 The horizontal effects on retaining walls of live load on carriageways and footpaths can be assumed to be zero when the fill is foamed concrete.

When fill comprises earthworks materials, the effects of live load induced earth pressures shall be taken into account.

4.5.4 Allowances shall be made for forces due to vehicle collision with parapets.

4.5.5 The effect of the forces from the base of the spandrel wall on the arch ring shall be taken into consideration.

Note: The deepest section of the spandrel wall adjacent to the pier or abutment wall may tend to span longitudinally to the abutment pier or to the ring in the next span and this behaviour may be considered in the design.

4.5.6 Where spandrel walls and their extensions in the form of wing walls extend for 15m or more, it will be necessary to consider the use of expansion joints at centres which will not normally exceed 10m. This may be avoided at the discretion of the designer if it is considered the walls have sufficient flexibility to take up the movement without excessive cracking. This normally applies to structures constructed with lime mortar.

4.6 Piers

4.6.1 Ultimate Limit State

4.6.1.1 At the ultimate limit state, piers shall be checked to ensure that collapse does not occur. The limitations of 4.3 apply. Where the height of a pier exceeds 12 times its thickness, the effect of displacements shall be considered.

4.6.1.2 The limiting stress state at the base of piers will often be governed by the foundations.

4.6.2 Serviceability Limit State

The piers shall be checked in accordance with 4.4.1 and 4.4.2.

4.7 Foundations

4.7.1 The bearing capacity of soils and fills shall be determined in accordance with the principles of soil mechanics. The design bearing capacity shall be determined from the design parameters for the soil or fill material in accordance with BD 74 (DMRB 2.1.8).

4.7.2 Foundation displacements and rotations shall be limited so as not to cause serviceability or ultimate limit state failures of the arch ring.

4.7.3 Where appropriate foundation design should take into consideration the recommendations of BA 59 (DMRB 1.3.6).

4.8 Parapets

4.8.1 Unless approval is given otherwise by the Overseeing Organisation, masonry parapets shall not be used on structures supporting or likely to affect trunk roads or motorways.

4.8.2 Parapets shall be in accordance with the Overseeing Organisation’s requirements.

4.8.3 Unless approval is given otherwise by the Overseeing Organisation, parapets shall be supported by an independent foundation of sufficient mass and extent to resist forces specified in Clause 6.7 of Appendix A of BD 37 (DMRB 1.3.14), without bearing directly onto spandrel or wing walls.

4.8.4 The principles are similar to parapets for reinforced soil retaining walls and possible arrangements for supporting parapets are given in BD 70 (DMRB 2.1.5).

4.8.5 Reinforced concrete members shall be designed in accordance with BS 5400: Part 4, as implemented by BD 24 (DMRB 1.3.1).

4.8.6 For bridges not supporting or affecting trunk roads or motorways, unreinforced masonry parapets may be used at the discretion of the Overseeing Organisation. Where unreinforced masonry parapets are used, the parapets should be designed in accordance with BS 6779: Part 4. They may be formed as extensions to spandrel walls.

5. REFERENCES

5.1 Design Manual for Roads and Bridges

Volume 1: Section 3: General Design

BD 24 - The Design of Concrete Highway Bridges and Structures. Use of BS 5400: Part 4:1990 (DMRB 1.3.1)

BD 37 - Loads for Highway Bridges (DMRB 1.3.14)

BD 57 - Design for Durability (DMRB 1.3.7)

BA 59 - Design of Highway Bridges for Hydraulic Action (DMRB 1.3.6)

BD 60 - The Design of Highway Bridges for Vehicle Collision Loads (DMRB 1.3.5)

Volume 2: Section 1: Substructures

BD 30 - Backfilled Retaining Walls and Bridge Abutments (DMRB 2.1)

BD 70 - Stengthened/Reinforced Soil and Other Fills for Retaining Walls and Bridge Abutments. Use of BS 8600:1995 (DMRB 2.1.5)

BD 74 - Foundations (DMRB 2.1.8)

Volume 3: Section 4: Assessment

BD 21 - The Assessment of Highway Bridges and Structures (DMRB 3.4.3)

BD 86 - The Assessment of Highway Bridges and Structures for the effects of Special Types General Order (STGO) and Special Order (SO) Vehicles (DMRB 3.4.19)

Volume 6: Section 1: Links

TD 27 - Cross-Sections and Headrooms (DMRB 6.1.2)

5.2 Manual Of Contract Documents For Highway Works

Volume 1: Specification for Highway Works (MCHW1)

5.3 British Standards

BS EN 1990:1994 - Eurocode: Basis of Design

BS EN 1991 - Eurocode 1: Actions on structures

BS EN 1991-1-1:2002	Part 1-1: General actions - Densities, self-weight, imposed loads for buildings
BS EN 1991-1-4	Part 1-4: General actions - Wind actions (not yet published)
BS EN 1991-1-5:2003	Part 1-5: General actions - Thermal actions
BS EN 1991-2:2002	Part 2: Traffic loads on bridges

BS EN 1996 - Design of Masonry Structures

BS 5400: Steel, Concrete and Composite Bridges:

Part 4: 1990: Code of Practice for Design of Concrete Bridges

BS 5628 Code of Practice for Use of Masonry:

Part 1: Structural use of reinforced masonry

BS 6779 Part 4 - Highway Parapets for Bridges and Other Structures: Specification for Vehicle Containment: Masonry Parapets

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5.4.1 Melbourne, C. and Gilbert, M. The behaviour of multi-ring brickwork arch bridges *The Structural Engineer*, Vol. 73. No 3. 7 Feb 1995 pp3 9-47.

5.4.2 Mair, A.J. A New UK Design Standard for Unreinforced Arch Bridges. Paper presented at First International Conference on Arch Bridges, Bolton, 3-6 September 1995, but not included in published proceedings.

5.4.3 Jackson, P.A. The stress limits for reinforced concrete in BS 5400. *The Structural Engineer.* Vol. 65A, No. 7. July 1987. pp. 9-17.

5.4.4 Choo, B.S. and Hogg, V. *Determination of the serviceability limit state in arches.* Arch bridges. Melbourne C Ed. Thomas Telford. 1995. pp 529-536.

5.4.5 Cox, D. and Halsall, R. *Brickwork Arch Bridges*. The Brick Development Association 1996.

5.4.6 Owen, D.R.J., Peric, D., Petrinic, N., Brookes, C.L. and James, P.J., Finite/Discrete Element Models for Assessment and Repair of Masonry Structures, Second International Arch Bridge Conference, Venice, Italy, October 1998.

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ANNEX A BACKGROUND INFORMATION

A1 Introduction

Experience shows that masonry arch bridges are a very durable form of construction and BD 57 (DMRB 1.3.7) says that their use should be considered. However, there has not previously been a standard for the design of new unreinforced arch bridges. The objective of this Standard is to encourage the renaissance in arch building using unreinforced masonry materials. It is based on an earlier draft discussed by Mair[5.4.2] but extensive changes have been required.

One reason for the changes is that this document was prepared as the EN (Euronorms) standards for structures and bridges were in the late stages of preparation. Although there are no proposals to introduce a bridge section to the masonry EN, and therefore masonry arch bridges will not be explicitly included in the new system, it was decided to bring this BD into line with ENs as far as possible. However, since some of the key standards and their National Annexes were not due for publication until after this document, it was necessary to include relevant sections rather than referring to the ENs.

This document is essentially a code of practice. However, so few new masonry arches have been built in recent years that there is no clear agreed practice. In some areas this document defines the approach to be followed. However, there are some areas where it has not been possible to define the best approach. The following identifies these areas and discusses the issues. It may be possible to add more definitive guidance later if agreed practice becomes established.

The document has a wide scope encompassing all unreinforced masonry arches which are likely to be built. However, in common with other codes of practice, it does not claim to give all the information required to design any structure within its scope.

The scope of earlier drafts included unreinforced concrete arches. However, there is a problem with these in that modern standards require reinforcement to control cracks, particularly early thermal cracks. This reinforcement detracts from the durability advantages of unreinforced structures. It also seemed unlikely that it would be economic, once such reinforcement was provided, to design structurally without using it. If the reinforcement is used structurally, such structures can be (and have been) designed using normal concrete standards such as BS 5400: Part 4. The most satisfactory solution may be to develop alternative ways of controlling early thermal cracking, such as using fibre reinforced concrete. However, there are no Departmental Standards for this approach. Unreinforced mass concrete structures were therefore removed from the scope. It is, however, recognised that if the above problems are solved, the document provides much of relevance to the design of mass concrete arches. The scope does include masonry arches with concrete bricks, blocks or indeed voussoirs.

The basic scope is unreinforced masonry arch bridges and, to maintain the full durability advantages, the documents aims to encourage structures with literally no reinforcement. However, it would still contain much of use to the design of arches with some reinforcement.

The following broadly follows the Chapters of the main text of the BD.

A2 Design Principles and Objectives

The standard follows limit state principles, as described in BS EN 1990, with separate partial factors for loads and materials and separate checks for ultimate and serviceability limit states. However, two differences from the approach in BS 5400 will be noted.

First, the document follows the BS EN 1990/ BS EN 1991 approach to loads. This means that there is no separate γ_{f3}. The values of the individual load factors have been increased to compensate. The approach to load combinations is also different. In principle, each transient load type is considered in turn at full value in combination with other load types at their reduced combination value where they are multiplied by the combination factor, ψ. In practice, it has proved possible to minimise the need to consider secondary loads quantitatively so that it is usually only necessary to consider permanent and traffic loads.

The document potentially gives three approaches to considering serviceability. One, which is inherited from the first draft, is "rigorous assessment" which is not fully defined. It is not clear there is a truly rigorous direct way of assessing serviceability at present but it was felt desirable to leave this possibility open. The next approach is to check stresses in the masonry calculated from an elastic analysis. Because the corresponding analysis has to assume the masonry takes no tensile stress, this analysis has to be non-linear. It is not in principle difficult but current standard arch analysis programs (which were written for assessment rather than design) do not enable it to be done. It is also debatable how valid the criteria are. Jackson[5.4.3] has argued that serviceability criteria for reinforced concrete structures are essentially arbitrary rules which have been found to give satisfactory results in the past. The arches currently in existence were not designed by such approaches and therefore SLS criteria for masonry arches do not have this fundamental basis.

The third approach is to use increased load factors at ULS. This approach is based on BD 21 (DMRB 3.4.3). In drafting this it was decided to base serviceability requirements for arches on increasing ultimate load factors so that an overall factor of safety of 2.0 was achieved. This was based on the observation that no permanent damage was observed below half failure load in tests. The factor of 2.0 has been confirmed as reasonable by more recent work[5.4.4]. This found that repeated loads of half ultimate strength did not cause problems in masonry unless it was saturated. An assessment code for masonry arches may need to consider saturated masonry but a design one does not. It can specify drainage to avoid the problem.

A major issue in the design of arches is the treatment of movements. There are two aspects to this. One is whether or not it is necessary to provide expansion joints in the spandrel walls. The other is whether or not it is necessary to give quantitative consideration of temperature effects and foundation movement in design.

Arch bridges which have been built recently[5.4.5] have generally been provided with expansion joints in the spandrel walls. However, many older bridges, including multi span viaducts, have given satisfactory service without these. One factor which is undoubtedly significant in this and in tolerance of movement generally is the difference between lime mortar used in earlier structures and cement mortar used in modern practice. The lower tensile strength of lime mortar tends to mean that movement is taken up in numerous small cracks which are not regarded as significant, whereas with cement mortar the cracks tend to be much less numerous but more serious. The document does leave the possibility of using lime mortar open. This is recommended if an existing bridge with lime mortar was being widened or worked on in any way as the different movement characteristics of cement and lime mortar masonry would lead to problems. However, the weakness and potentially poorer durability of lime mortar has discouraged its use in modern structures. One problem may be that the purity of modern lime means that modern lime mortar is actually weaker and has less of a set than traditional lime mortar. A possible solution would be to use a cement lime mortar, which may actually be more comparable in structural characteristics with traditional lime mortar. However, unfortunately, lack of experience has meant it has not been possible to define specific recommendations for consideration of movement and movement joints in structures with this type of mortar.

Treatment of foundation movements is an area where there is no agreed practice. When an earlier draft of the BD was circulated for comment, one comment was that it was not advisable to use arches, at least with cement mortar, without piled foundations unless the ground conditions were very favourable, e.g. rock. Another comment was that piles should not be needed and arches were insensitive to movement. Analysis does, however, show that the sensitivity of arches to movements increases rapidly as the span to rise ratio increases. The standard therefore requires explicit consideration of movements (both foundation and temperature) where the span to rise ratio is large but otherwise leaves this open to judgment. The standard does require foundations to be designed to normal modern soil mechanics principles. Many older arches have foundations which appear inadequate to modern standards and therefore this requirement should reduce the potential for problems to be caused by foundation movements.

A3 Actions

The treatment of permanent action is consistent with BS EN 1990 and 1991. As this BD was drafted before the National Annex for the relevant parts of these, the factors are reproduced in full. The same would apply to the factors for thermal effects. However, since these will rarely need to be considered quantitatively, it was decided to leave reference to the Overseeing Organisation. The standard does provide the additional information needed to obtain the actual temperatures in arch structures from BS EN 1991-1-5.

It seems unlikely that wind will ever be critical in a masonry arch bridge so no detailed provisions are given.

Current traffic loading representing normal traffic in both BD 37 (DMRB 1.3.7)/BS 5400: Part 2 and BS EN 1991-2 was derived primarily with beam and slab type bridges in mind. Studies show that it is not necessarily valid for masonry arches. Real vehicle loads are therefore considered. The vehicles are largely the same as those in BD 21 (DMRB 3.4.3) but the 10% contingency factor as in BD 37 (and also UK calibration of BS EN 1991-2) is added in. This leads to the need for load factors which are not the same as in BS EN 1991.

Unlike BD 21 (DMRB 3.4.3) the document does not consider axle lift off. This is consistent with both BD 37 and BS EN 1991-2 and reflects the fact the document is meant for new construction where the highway alignment is to modern standards. If the standard was being used for work on existing structures where this does not apply, the provisions in BD 21 could be used.

The treatment of abnormal vehicles is based on the draft National Annex (NA) for BS EN 1991-2. However, as it was anticipated that this BD would be published before that document it was necessary to reproduce the clauses. It was also necessary to re-introduce a vehicle from BD 86 (DMRB 3.4.19) that is not in the draft BS EN 1991-2 NA. This was because it gives a more severe axle. This was not needed in the BS EN 1991-2 NA because BS EN 1991-2's very severe Load Model 1 tandem axles and Load Model 2 single axle covered the effects.

Much of the traffic loading section is concerned with the treatment of multiple lanes. In practice at present most arch analysis is done using two-dimensional approaches and therefore will not be affected by this. However, it was considered the standard should cover the loading required if a three-dimensional analysis is used.

A4 Design and Resistances

The document leaves wide choice of analytical approach for checking the ring at ULS. There is, however, no provision for the use of MEXE. Although this is known to be conservative for long span arches, experience shows it is often less conservative than other methods for short span structures, particularly if they have significant fill over the crown. Assessment using the more sophisticated discrete element approach[5.4.6] did not identify a satisfactory explanation of the extra strength from MEXE and even sometimes gave **lower** strengths than mechanism methods. It was not therefore considered safe to recommend the approach.

The code allows the characteristic strength of the material to be determined from BS 5628. There is a problem during the transition from BSs to ENs. The ENs for bricks have already been published and they supersede the previous BSs. However, they calculate brick strength on a different basis from the BSs. This results in straightforward use of BS 5628 with the ENs being unsafe. Since it is anticipated that it will be some time before EN 1996 is fully in use for brickwork design, it is proposed to publish a new BS 5628 which is compatible with the brick ENs. This will enable this BD to refer to BS 5628 and the brick ENs. However, in the interim, to avoid unsafe results, the document continues to refer to the BSs for bricks.

The material partial factor used is the same as in the original draft. It is lower than in BS 5628. The document also uses a rectangular stress block with the material all at the design strength. The assumed constant stress has been reduced from 0.66 in the draft to 0.6 f_{ck} which is consistent with the treatment of concrete. In principle an analysis that used a realistic stress strain relationship could revert to use of 0.66. The design strength is still higher than in BS 5628. However, BD 21 (DMRB 3.4.3) absorbs all the safety factors into the load factor and uses a higher design strength. Many arch structures are insensitive to material strength but it appears that for cases where sensitivity is relatively high, this BD will tend to be more conservative than BD 21.

The check on shear is not always done in assessment but appears to be required theoretically. A case of a bridge with damage apparently due to being close to this form of failure has been observed.

The treatment of serviceability has already been discussed in A2 above. It is anticipated that the approach of increasing the load factors at ULS will most commonly be used. Where stresses are checked, there is also a requirement to limit the eccentricity of the line of thrust. This is equivalent to crack width checks in concrete.

The spandrel walls are checked much as any other retaining walls whilst the piers are checked on the same basis as the ring. The document requires the effect of the forces on the spandrel walls to the ring to be considered. This is significant as many existing arches have suffered damage due to these effects. This is particularly common in structures where there is a continuous vertical plane of mortar joints in the ring near the critical section, as in some structures where stone (real or reconstituted) voussoirs are used on the face of an otherwise brick ring. This detail is not advised. This does not preclude use of voussoir faces in otherwise brick rings as the continuous vertical mortar joint can be avoided by using alternate wider and narrower voussoirs so they can be bonded into the brickwork.

On some recently constructed bridges (such as Kimbolton Butts[5.4.5]) the issue has been avoided by using reinforced spandrel walls. At present the BD does not encourage this but it does not define how to check unreinforced rings for the effect of forces on the spandrel wall. BS 5628 does allow a nominal tension in brickwork in flexure and it would seem the only quantitative design approach to the effect of spandrel forces on the ring is to use this in the ring. Its use is not normally recommended in direct tension except for certain accidental cases and even then half the flexural value is used. However, combined direct tension and flexure is limited to the full value. If the combined stress is calculated ignoring the beneficial effect of the curvature of the ring it seems reasonable to use the tensile strength given by BS 5628 although in this case the normal BS 5628 material partial factor should be used rather than the lower one used for compression in Chapter 3 of the BD. In reality, the tensile strength in the relevant direction is enhanced because of the increased shear strength due to the compression in the ring. It may be possible to devise a design approach which uses this but there is no standard agreed method. If the compression is high the point would eventually be reached where the tensile strength of the units governed the tensile strength of the masonry.

Foundations will be designed to normal soil mechanics principles. As noted in A2 above, some judgement is required in deciding whether the foundation movements are significant to arch stresses.

Parapets were one of the most discussed items. At present, masonry parapets are not allowed on bridges on or likely to affect motorways and trunk roads in England. For such bridges, separate structures will be required to support the parapets as was required by the original draft. This is similar to the approach used for reinforced earth retaining walls and reference to the relevant design standard is made. However, the great majority of the arches, the satisfactory behaviour of which led to moves to introduce this standard, have masonry parapets forming simple extensions to the spandrel walls. It therefore seemed inappropriate to exclude this form of construction for all bridges. It also seemed likely that the separate parapets would detract from the aesthetic appeal of arches and also from the economy particularly of short span structures. Therefore, this form of construction is allowed at the discretion of the Overseeing Organisation.