Incorporating Corrigendum No. 1

NATIONAL ANNEX

UK National Annex to Eurocode 1: Actions on structures –

Part 2: Traffic loads on bridges

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National Annex (informative) to BS EN 1991-2:2003, Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges

Introduction

This document has been prepared by BSI Subcommittees B/525/1, *Actions (loadings) and basis of design*. In the UK it is to be used in conjunction with BS EN 1991-2:2003.

NA.1 Scope

This document gives:

a) the UK decisions for the Nationally Determined Parameters described in the following subclauses of BS EN 1991-2:2003:

- **1.1** (3)
- **2.2** (2) Note 2
- **2.3** (1) Note and (4) Note
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b) the UK decisions on the status of BS EN 1991-2:2003 informative annexes;

c) references to non-contradictory complementary information.

NA.2 Nationally determined parameters

NA.2.1 Complementary conditions [BS EN 1991-2:2003, 1.1 (3)]

The models given in **NA.2.34** and **NA.3.1** should be used for the design of buried structures, retaining walls and tunnels, subject to road traffic loading.

NA.2.2 Infrequent values of loads [BS EN 1991-2:2003, 2.2 (2) Note 2]

Infrequent values of loading should not be used.

NA.2.3 Appropriate protection against collision [BS EN 1991-2:2003, 2.3 (1)]

The requirements for protection against collision from road and rail traffic should be determined for the individual project. See also **NA.4**.

NA.2.4 Impact forces due to boats, ships or aeroplanes [BS EN 1991-2:2003, 2.3 (4)]

For impact forces due to boat and ship impacts, refer to BS EN 1991-1-7 and its National Annex.

NA.2.5 Bridges carrying both road and rail traffic [BS EN 1991-2:2003, 3 (5)]

The rules for bridges intended for both road and rail traffic should be determined for the individual project and should be based on, where appropriate, the load models for road and rail traffic as defined in BS EN 1991-2 and this National Annex.

NA.2.6 Models for loaded lengths greater than 200 m [BS EN 1991-2:2003, 4.1 (1) Note 2]

Load Model 1 may be used for loaded lengths up to 1 500 m.

NA.2.7 Weight restricted bridges [BS EN 1991-2:2003, 4.1 (2)]

For road bridges where effective means are provided to strictly limit the weight of any vehicle, specific load models may be determined for the individual project.

NA.2.8 Complementary load models [BS EN 1991-2:2003, 4.2.1 (1)]

Complementary load models and rules for their application may be determined for the individual project. See also **NA.2.34**.

NA.2.9 Models for special vehicles [BS EN 1991-2:2003, 4.2.1 (2)]

Complementary load models for special vehicles and rules for their application may be determined for the individual project. See also **NA.3.1**.

NA.2.10 Conventional height of kerbs [BS EN 1991-2:2003, 4.2.3 (1)]

The minimum value of the height of a kerb for defining the carriageway width should be taken as 75 mm.

NA.2.11 Use of Load Model 2 [BS EN 1991-2:2003, 4.3.1 (2) (b)]

No additional information is provided.

NA.2.12 Adjustment factors α for Load Model 1 [BS EN 1991-2:2003, 4.3.2 (3) Notes 1 and 2]

The adjustment factors α for the Tandem System and the UDL should be taken from Table NA.1.

Table NA.1 **Adjustment factors α_Q and α_q for Load Model 1**

Location	α_Q for tandem axle loads	α_q for UDL loading
Lane 1	α_{Q1} = 1,0	α_{q1} = 0,61 (See note)
Lane 2	α_{Q2} = 1,0	α_{q2} = 2,2
Lane 3	α_{Q3} = 1,0	α_{q3} = 2,2
Other lanes	—	α_{qn}= 2,2
Remaining area	—	α_{qr} = 2,2
NOTE α_{q1} should be taken as 1,0 for 4.4.1(2) of BS EN 1991-2		

NA.2.13 Use of simplified alternative Load Models [BS EN 1991-2:2003, 4.3.2 (6)]

The simplified alternative load models given should not be used.

NA.2.14 Adjustment factor β for Load Model 2 [BS EN 1991-2:2003, 4.3.3 (2)]

The recommended value for β_Q should be used.

NA.2.15 Wheel contact surface for Load Model 2 [BS EN 1991-2:2003, 4.3.3 (4)]

The contact surface of each wheel in Load Model 2 should be taken as a square of sides 0.40 m.

NA.2.16 Load Model 3 (Special Vehicles) [BS EN 1991-2:2003, 4.3.4 (1)]

The following defines Load Model 3 and its conditions of use. They do not describe actual vehicles but have been calibrated so that the effects of the nominal axle weights, multiplied by the Dynamic Amplification Factor, represent the maximum effects that could be induced by actual vehicles in accordance with the Special Types General Order (STGO) and Special Order (SO) Regulations.

The choice of the particular STGO or SO model vehicle for the design of structures on motorways, trunk roads and other minor roads should be determined for the individual project.

NA.2.16.1 Basic models for STGO vehicles

The following three *SV* model vehicles simulate vertical effects of different types of STGO vehicles with nominal axle weights not exceeding 16,5 tonnes.

NA.2.16.1.1 *SV80*

The *SV80* vehicle is intended to model the effects of STGO Category 2 vehicles with a maximum gross weight of 80 tonnes and a maximum basic axle load of 12,5 tonnes. Figure NA.1(a) gives the basic axle loads, the plan and axle configuration for the *SV80* vehicle.

NA.2.16.1.2 *SV100*

The *SV100* vehicle is intended to model the effects of STGO Category 3 vehicles with a maximum gross weight of 100 tonnes and a maximum basic axle load of 16,5 tonnes.

Figure NA.1(b) gives the basic axle loads, the plan and axle configuration for the *SV100* vehicle.

NA.2.16.1.3 *SV196*

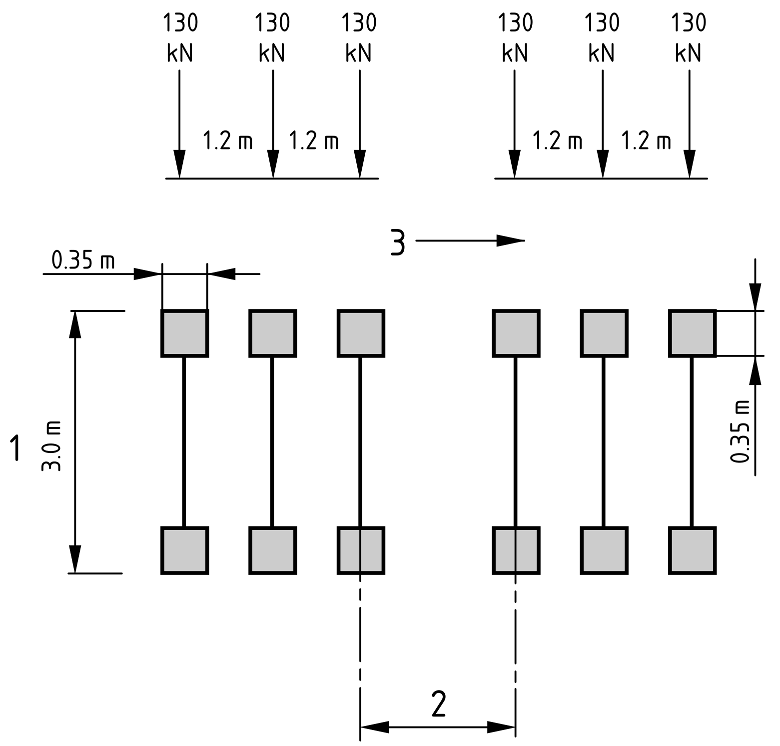
The *SV196* model represents the effects of a single locomotive pulling a STGO Category 3 load with a maximum gross weight of 150 tonnes and a maximum basic axle load of 16,5 tonnes with the gross weight of the vehicle train not exceeding 196 tonnes.

Figure NA.1(c) gives the basic axle loads, the plan and axle configuration for the *SV196* vehicle.

The wheel loads of all the three *SV* model vehicles should be uniformly distributed over a square contact area as shown in Figure NA.1.

Figure NA.1 **Basic longitudinal configuration of *SV* model vehicles**

(a) *SV80* Vehicle



Key

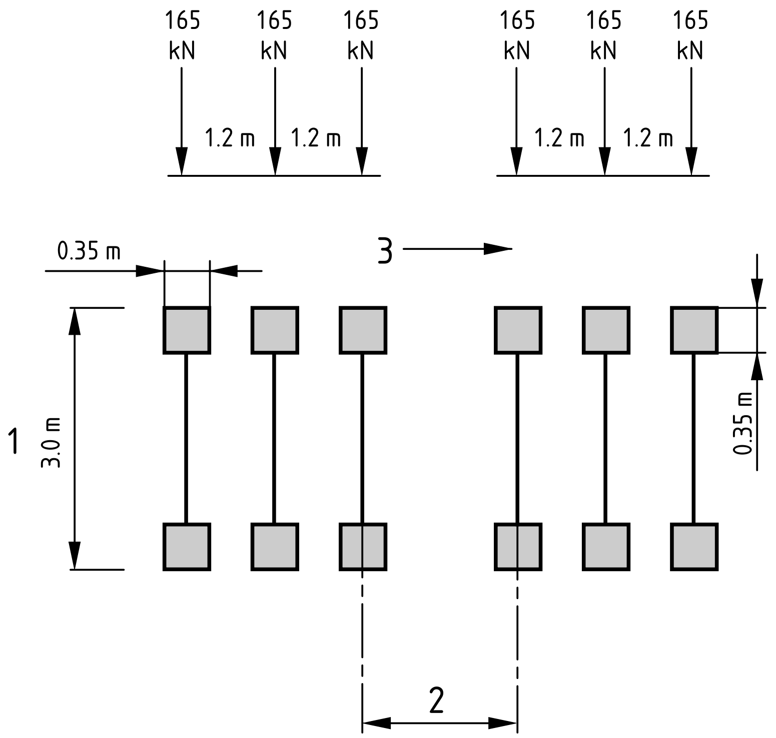
1 = Outside track and overall vehicle width

2 = Critical of 1.2 m or 5.0 m or 9.0 m

3 = Direction of travel

Figure NA.1 **Basic longitudinal configuration of *SV* model vehicles** (*continued*)

(b) *SV100* Vehicle



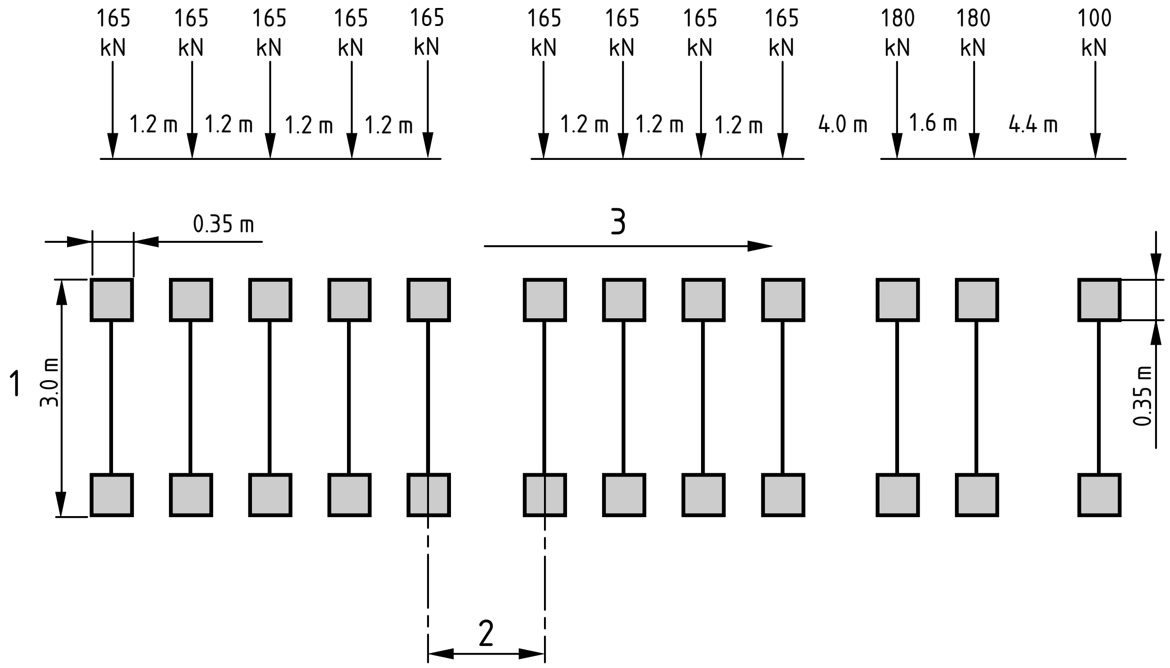
Key

1 = Outside track and overall vehicle width

2 = Critical of 1.2 m or 5.0 m or 9.0 m

3 = Direction of travel

(c) *SV196* Vehicle



Key

1 = Outside track and overall vehicle width

2 = Critical of 1.2 m or 5.0 m or 9.0 m

3 = Direction of travel

NA.2.16.2 Basic models for Special Order Vehicles

The following four *SOV* model vehicles simulate vertical effects of Special Order (SO) vehicles with trailer weights limited to:

i) *SOV-250* – Maximum total weight of SO trailer units up to 250 tonnes

ii) *SOV-350* – Maximum total weight of SO trailer units up to 350 tonnes

iii) *SOV-450* – Maximum total weight of SO trailer units up to 450 tonnes

iv) *SOV-600* – Maximum total weight of SO trailer units up to 600 tonnes.

The longitudinal configuration of the four model vehicles is shown in Figure NA.2. The standard configuration has a trailer with two bogies and two tractors; one pulling and one pushing. However, on structures located on a stretch of road with a gradient steeper than 1 in 25, six tractor units in any combination of pulling and pushing that produces the worst effect, should be used for design.

Figure NA.2 **Basic longitudinal configuration of *SOV* model vehicles**

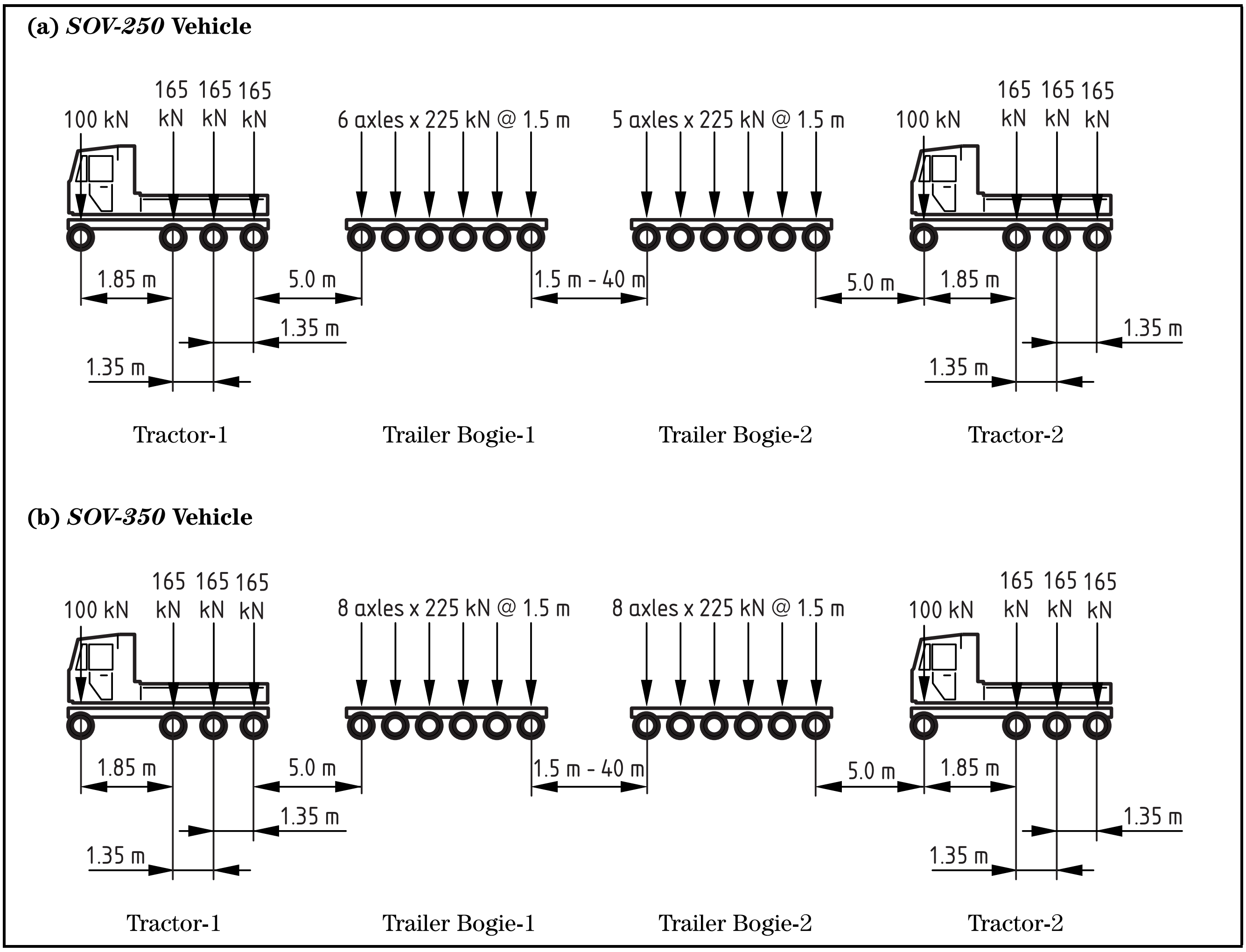
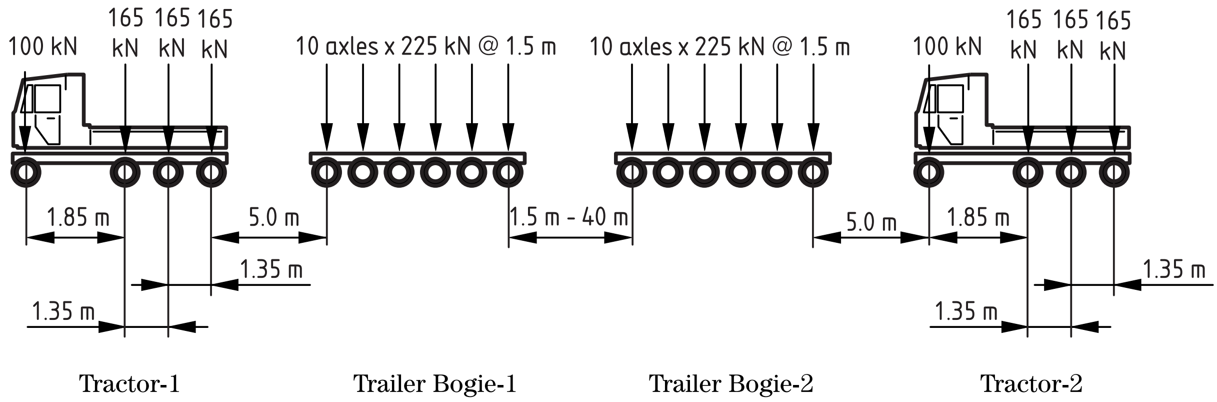
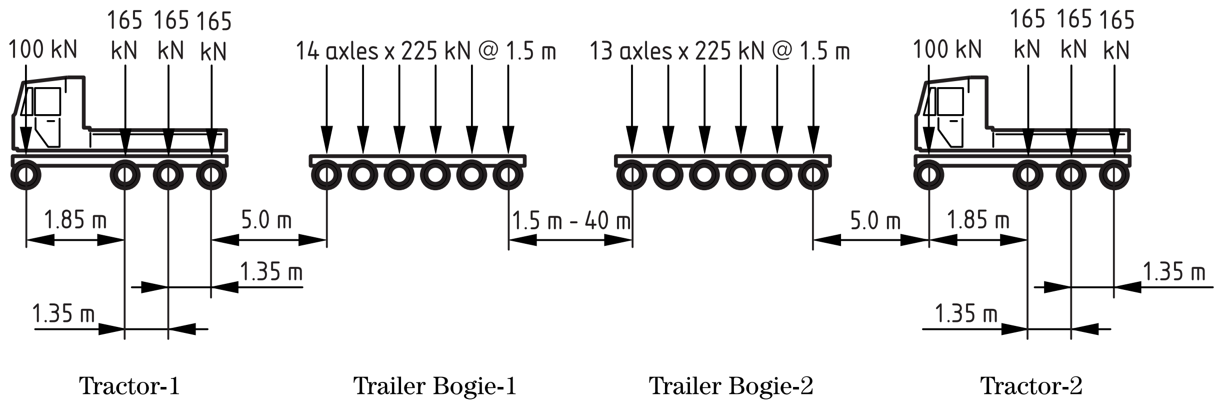


Figure NA.2 **Basic longitudinal configuration of *SOV* model vehicles** (*continued*)

(c) *SOV-450* Vehicle



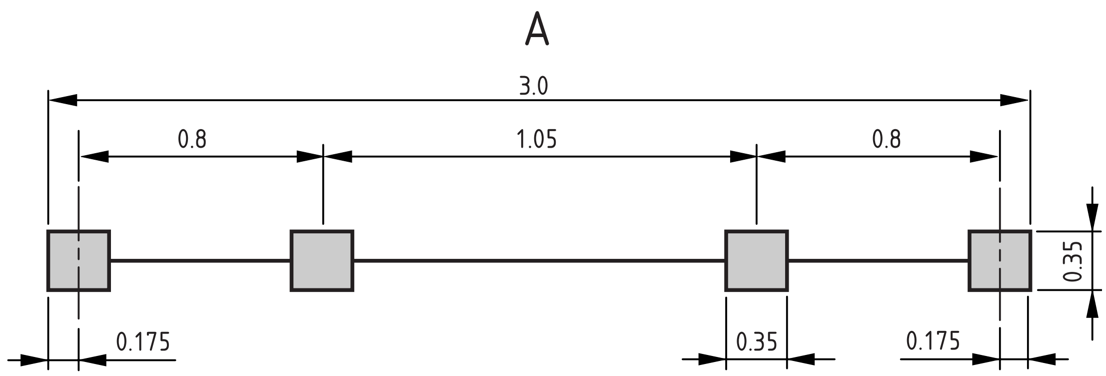
(d) *SOV-600* Vehicle



NOTE For simplicity, 6-axle trailer bogies are shown. The actual number of axles of trailer bogie should be that stated above the figure.

The lateral wheel arrangement for the trailer axles of all the *SOV* model vehicles is shown in Figure NA.3. All the wheels are of equal weight. The contact surface of each wheel should be taken as a square of sides 0,35 m.

Figure NA.3 **Lateral wheel arrangement for trailer axles of all *SOV* models**



Key

A = Outside track and overall vehicle width, 3,0 m

The tractor axles of the model vehicles have two wheels, each of equal weight and with square contact areas of side 0,35 m. The outside track and overall width of the vehicle is 3,0 m.

NA.2.16.3 Dynamic amplification factors

In determining the load effects of *SV* and *SOV* vehicles, the basic axle loads given in Figures NA.1 and NA.2 should be multiplied by the appropriate Dynamic Amplification Factor (*DAF*) for each axle as given in Table NA.2, depending on the value of the basic axle load.

Table NA.2 **Dynamic Amplification Factors for the *SV* and *SOV* vehicles**

Basic axle load	DAF
100 kN	1,20
130 kN	1,16
165 kN	1,12
180 kN	1,10
225 kN	1,07

NA.2.16.4 Application of special vehicle models on the carriageway

The *SV* or *SOV* vehicle loading should be combined with Load Model 1, given in **4.3.2** of BS EN 1991-2, together with the load adjustment factors given in **NA.2.12** as follows.

i) Only one *SV* or *SOV* model vehicle should be considered on any one superstructure.

ii) The Load Model 1 should be considered to be at the "frequent" values as defined in **4.5** of BS EN 1991-2 and in BS EN 1990, Annex A.2 and its National Annex. The loading should be applied to each notional lane and the remaining area of the bridge deck.

The *SV* or *SOV* vehicle can be placed at any transverse position on the carriageway, either wholly within one notional lane or straddling two adjacent lanes, with its side parallel to the kerb. The *SV* or *SOV* vehicle should be placed at the most unfavourable position transversely and longitudinally over the loaded length, in order to produce the most severe load effect at the section being considered. The *SV* or *SOV* vehicle should be applied on influence lines in its entirety and should not be truncated.

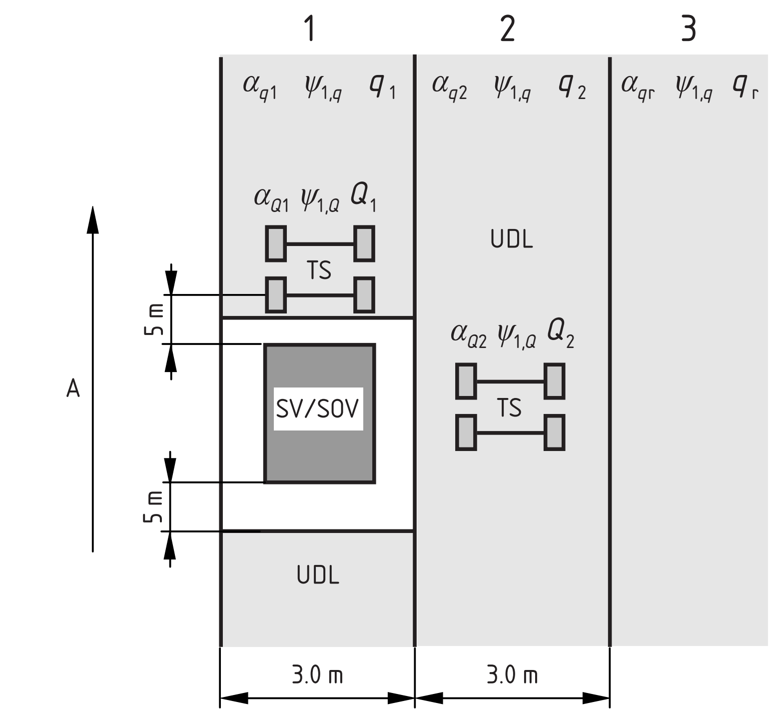
Where the *SV* or *SOV* vehicle lies fully within a notional lane the associated Load Model 1 loading should not be applied within 5 m from the centre of outermost axles (front and rear) of the *SV* or *SOV* vehicle in that lane as illustrated in Figure NA.4.

Where the *SV* or *SOV* vehicle lies partially within a notional lane and the remaining width of the lane, measured from the side of the *SV* or *SOV* vehicle to the far edge of the notional lane, is less than 2,5 m [see Figure NA.5(a)], the associated Load Model 1 loading should not be applied within 5 m of the centre of the outermost axles (front and rear) of the *SV* or *SOV* vehicle in that lane.

Where the *SV* or *SOV* vehicle lies partially within a notional lane and the remaining width of lane, measured from the side of the *SV* or *SOV* vehicle to the far edge of the notional lane, is greater than or equal to 2,5 m [see Figure NA.5(b)], the "frequent" value of the uniformly distributed load of the Load Model 1 may be applied over the remaining width of the notional lane (in addition to remaining parts of the lane). The "frequent" value of the tandem system for that notional lane may be applied anywhere along its length.

On the remaining lanes not occupied by the *SV* or *SOV* vehicle, the Load Model 1 at its "frequent" value should be applied in accordance with **4.3.2** of BS EN 1991-2.

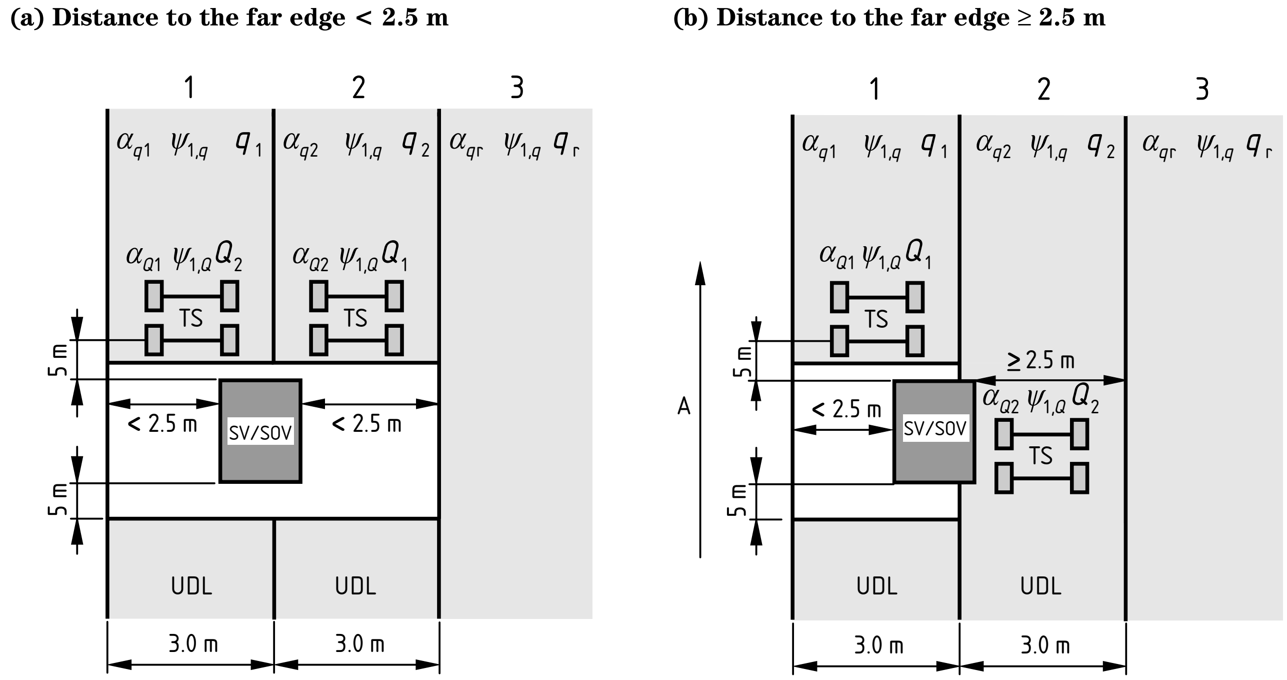
Figure NA.4 **Typical application of *SV* or *SOV* and Load Model 1 loading when the *SV* or *SOV* vehicle lies fully within a notional lane**



Key

A = Direction of travel 1 = Lane 1 2 = Lane 2 3 = Remaining area

Figure NA.5 **Typical application of *SV* or *SOV* and Load Model 1 loading when the *SV* or *SOV* vehicle straddles two adjacent lanes**



Key

A = Direction of travel 1 = Lane 1 2 = Lane 2 3 = Remaining area

The notional lanes are located so as to produce the maximum load effect at the part of the structure under consideration in accordance with **4.2.4** of BS EN 1991-2.

NA.2.17 Upper limit of the braking force on road bridges [BS EN 1991-2:2003, 4.4.1 (2)]

The upper limit for the braking force should be taken as 900 kN.

NA.2.18 Horizontal forces associated with Load Model 3 [BS EN 1991-2:2003, 4.4.1 (3)]

NA.2.18.1 Longitudinal braking and acceleration forces

The longitudinal force should be taken as the more severe of either the braking or the acceleration force, determined as below.

The characteristic value of longitudinal braking force on individual axles, $Q_{lk,S}$, expressed in kN, of special vehicles (both *SV* and *SOV*) should be calculated as follows:

$$Q_{lk,S} = \delta w$$

Where δ is the deceleration factor and w is the basic axle load of the relevant *SV* or *SOV* vehicle in *kN* shown in Figures NA.1, NA.2 and NA.3. The value of δ should be taken as 0,5 for *SV80*, 0,40 for *SV100*, 0,25 for the *SV196* and 0,20 for all of the *SOV* model vehicles.

The acceleration force should be taken as 10% of the gross weight of the *SV* or *SOV* vehicle and distributed between the axles and wheels in the same proportion as the vertical loads.

NA.2.18.2 Centrifugal force

The characteristic value of centrifugal force from *SV* or *SOV* vehicles, $Q_{tk,S}$, should be calculated as follows and applied in a manner similar to Q_{tk} for normal traffic as given in BS EN 1991-2:2003, **4.4.2**.

$$V = \rho\{\text{ whichever is greater: 30 or } \sqrt{\frac{100 \times g \times r}{r + 150}}\ \} \leq V_{Limit}$$

$$Q_{tk,S} = \frac{W \times V^2}{g \times r}$$

Where:

V = velocity of the *SV* or *SOV* vehicle in m/sec

V_{Limit} = speed limit on the road in m/sec

W = weight of the *SV* or *SOV* vehicle in kN

r = radius of curvature in m

g = acceleration due to gravity = 9.8 m/sec^2

ρ = 0,86 for *SV80*, 0,77 for *SV100*, 0,55 for *SV196*, 0,41 for *SOV 250*, 0,36 for *SOV 350*, 0,33 for *SOV 450* and 0,30 for *SOV 600*.

The centrifugal force should be distributed between axles and wheels in the same proportion as the vertical loads.

NA.2.19 Horizontal force transmitted by expansion joints or applied to structural members [BS EN 1991-2:2003, 4.4.1 (6)]

The recommended value should be used.

NA.2.20 Lateral forces on road bridge decks [BS EN 1991-2:2003, 4.4.2 (4)]

The minimum transverse force due to skew braking or skidding, Q_{trk}, should be taken as 50% of the longitudinal braking force, Q_{lk}, for loaded lengths up to 120 m. For loaded lengths greater than or equal to 120 m a transverse force of 280 kN should be used.

NA.2.21 Groups of traffic loads [BS EN 1991-2:2003, 4.5.1 Table 4.4a Notes a) and b)]

The groups of traffic loads should be taken as defined in Table NA.3 instead of Table 4.4a of BS EN 1991-2.

Table NA.3 **Assessment of groups of traffic loads (characteristic values of the multi-component action)**

Load type		**Carriageway**						**Footways and cycletracks**
		Vertical forces				**Horizontal forces**		**Vertical forces only**
Reference		**4.3.2**	**4.3.3**	Annex A	**4.3.5**	**4.4.1**	**4.4.2**	**5.3.2.1** Equation (5.1)
Load system		LM1 (TS and UDL)	LM2 (Single axle)	LM3 (Special vehicles)	LM4 (Crowd loading)	Braking and acceleration forces	Centrifugal and transverse forces	Uniformly distributed load
Groups of loads	**gr1a**	Characteristic						0.6 times Characteristic
	gr1b		Characteristic					
	gr2	Frequent(4)				Characteristic	Characteristic	
	gr3(1)							Characteristic
	gr4				Characteristic			Characteristic
	gr5	Frequent(4)		Characteristic				
	gr6			Characteristic		Characteristic	Characteristic	
	(shaded)	Dominant component action (the group is sometimes designated by this component for convenience).						

(1) This group is irrelevant if gr4 is considered

(2) Characteristic value obtained from **5.3.2.1**

(3) This is a reduced value accompanying the characteristic value of LM1 and should not be factored by ψ_1. However, when gr1a is combined with leading non-traffic actions this value should be factored by ψ_0

(4) The ψ_1 factors should be taken from the UK National Annex to BS EN 1990

NA.2.22 Conditions for use of Fatigue Load Models [BS EN 1991-2:2003, 4.6.1 (2) Note 2c]

There are no special conditions for the use of Fatigue Load Model 1.

Fatigue Load Model 2 should only be used for cases where the fatigue verification is not influenced by the simultaneous presence of several lorries on the bridge, unless account of their presence is taken using the following approach:

(i) Where bridge influence line lengths permit, the maximum and minimum stresses caused by Fatigue Load Model 2 should be obtained by considering the worst load effect of the most onerous vehicle accompanied in the same lane, with a 40 m clearance, by the lightest vehicle in Table 4.6 of BS EN 1991-2, if this causes a worse load effect.

(ii) Where two or more notional lanes influence the design detail, the maximum and minimum stresses should be obtained from Fatigue Load Model 2 by placing the most onerous vehicle on the most onerous part of the influence line in the most onerous lane, plus the lightest vehicle on the most onerous part of the influence line in one other lane.

NA.2.23 Definition of traffic categories and traffic flows [BS EN 1991-2:2003, 4.6.1 (3) Note 1]

Heavy goods vehicle numbers for use in fatigue design should be taken as indicated in Table NA.4 with the additional Notes 4 and 5. Heavy goods vehicle counts may be obtained from site surveys by doubling the observed number of lorries with three or more axles.

Table NA.4 **Indicative numbers of heavy goods vehicles expected per year and per lane in the United Kingdom**

	Traffic categories		**N_{obs} per lane (millions per year)**	
Type	**Carriageway layout**	**No. of lanes per carriageway**	**Each slow lane**	**Each fast lane**
Motorway	Dual	3	2.0	1.5
Motorway	Dual	2	1.5	1
All purpose	Dual	3		
All purpose	Dual	2		n/a
Slip road	Single	2		
All purpose	Single	3	1.0	0
All purpose	Single	2		0
Slip road	Single	1		n/a
All purpose	Single	2	0.5	0
Local (low lorry flow)	Single	2	0.05	0

NOTE 1 Notes 1 and 2 in BS EN 1991-2 may be disregarded for UK purposes.

NOTE 2 There is no general relation between traffic categories for fatigue verifications, and the loading classes and associated α factors mentioned in ***4.2.2*** *and* ***4.3.2****.*

NOTE 3 Intermediate values of N_{obs} are not excluded, but are unlikely to have significant effect on the fatigue life.

NOTE 4 Basing the numbers of heavy goods vehicles on counts of multi-axled lorries ensures a reasonably reliable match between the codified traffic model and the number and types of vehicle that cause the most fatigue damage in the actual traffic.

NOTE 5 The values presented in Table NA.4 are design values that are intended to reflect approximate road capacities, and they may not match observations of current usage. Traffic flows at a small number of sites may exceed these values, but the differences are unlikely to have a very significant influence on designs.

NA.2.24 Dynamic additional amplification factor due to expansion joints [BS EN 1991-2:2003, 4.6.1 (6)]

The recommended value should be used.

NA.2.25 Fatigue Load Model 3 [BS EN 1991-2:2003, 4.6.4 (3)]

The conditions of application for two vehicles in the same lane should be determined for the individual project.

NA.2.26 Fatigue Load Model 4 [BS EN 1991-2:2003, 4.6.1 (2) Note 2(e), 4.6.5 (1) Note 2]

As allowed in **4.6.5**(1) Note 2 and **4.6.1**(2) Note 2(e), the Fatigue Load Model 4 as defined below, along with the rules for its application, should be used in place of the model given in **4.6.5** of BS EN 1991-2.

Fatigue Load Model 4 may be used where the application of models 1, 2 and 3 all fail to provide sufficient fatigue life. Fatigue Load Model 4 may also be used when the influence line length, for details sensitive to fatigue, is short enough to have reversals of sign within a loaded length that is similar to typical vehicle wheel and axle spacings.

The standard lorries given in Table NA.5 for Fatigue Load Model 4 should be used for fatigue design on all routes in the UK. Where the length of the influence line permits, and/or where two or more notional lanes influence the design detail, Fatigue Load Model 4 should be applied as follows.

The fatigue damaging stress cycles due to the transit of Fatigue Load Model 4 lorries should be assessed and counted using the rainflow counting procedure described in BS EN 1993-1-9. Fatigue damage should be assessed on the basis of stress cycles calculated from two traffic lanes only. These lanes (described as lanes 1 and 2 for the purpose of this clause) are the two notional lanes that individually cause the most theoretical fatigue damage in the component under consideration. Vehicle numbers in these lanes should be obtained from Table NA.4.

Damage summation D_d is obtained by adding contributions from the following cases.

i) Lane 1 traffic alone, with 80% of lane 1 lorry numbers.

ii) 20% of lane 1 traffic running in convoy with vehicles at 40 m spacing, centre of rearmost axle of front vehicle to centre of foremost axle of vehicle behind.

iii) Lane 2 traffic alone, with 80% of lane 2 lorry numbers.

iv) 20% of lane 2 traffic running in convoy with vehicles at 40 m spacing, centre of rearmost axle of front vehicle to centre of foremost axle of vehicle behind.

The effect of side-by-side running should be allowed for by multiplying the total damage, $D_{d,}$ by factor K_b.Z, where: K_b = ratio of the maximum stress range caused by single vehicles in lane 2 to the maximum stress range caused by single vehicles in lane 1, and:

i) if loaded length ≤ 3,0 m, Z = 1,0;

ii) if 3,0 m < loaded length < 20 m, Z varies linearly in proportion to the logarithm of the loaded length from 1,0 to 1,5;

iii) if loaded length ≥ 20 m, Z=1,5.

Table NA.5 **Set of equivalent lorries for Fatigue Load Model 4**

Total axles	**Chassis type**	**Average spacings, m**	**Loading group**	**Total weight kN**	**Axle loads, kN**	**No in each group per million commercial vehicles**	**Vehicle Designation**
18	Girder trailer and 2 tractors	4.5 1.5 4.0 1.5 (5no.) 13.5 1.5 (5no.) 4.0 4.5 1.5	H	3680	80 160 160 240(6no.) 240(6no.) 80 160 160	10	18GT-H
			M	1520	80 160 160 60(6no.) 60(6no.) 80 160 160	30	18GT-M
9	Girder trailer and tractor	4.5 1.5 4.0 1.5 1.5 1.5 1.5 1.5	H	1610	70 140 140 210 210 210 210 210 210	20	9TT-H
			M	750	50 110 110 80 80 80 80 80 80	40	9TT-M
7	Girder trailer and tractor	4.5 1.5 4.0 2.0 10.0 2.0	H	1310	70 140 140 240 240 240 240	30	7GT-H
			M	680	60 130 130 90 90 90 90	70	7GT-M
	Articulated	3.0 1.5 9.5 1.5 1.5 1.5	H	790	70 100 100 130 130 130 130	20	7A-H
5	Articulated	3.0 1.5 9.5 1.5	H2	630	70 130 130 150 150	280	5A-H2
		3.5 5.5 1.5 1.5	H	380	70 100 70 70 70	90 500	5A-H
			M	300	50 70 60 60 60	90 000	5A-M
			L	190	40 60 30 30 30	90 000	5A-L
4	Articulated	3.0 6.5 1.5	H	240	40 80 60 60	45 000	4A-H
			M	175	40 55 40 40	45 000	4A-M
			L	145	35 50 30 30	45 000	4A-L
	Rigid	1.5 3.5 1.5	H	280	50 50 90 90	8 000	4R-H
			M	240	40 40 80 80	8 000	4R-M
			L	120	20 20 40 40	8 000	4R-L
3	Articulated					Not used	3A
	Rigid	4.0 1.5	H	240	60 90 90	10 000	3R-H
			M	195	45 75 75	10 000	3R-M
			L	120	60 45 45	10 000	3R-L
2	Rigid	4.0	H	135	50 85	170 000	2R-H
			M	65	30 35	170 000	2R-M
			L	30	15 15	200 000	2R-L

Key

O = Special axle. Applies to all vehicles over 5 axles with 2–8 tyres and outer track 2,4 m to 3,4 m. Specific vehicle axle arrangements are to be defined for the individual project.

◎ = Steering axle. 2 tyre, 2,0 m track

● = Standard axle. 4 tyre, 1,8 m track

NA.2.27 Fatigue Load Model 5 (based on recorded traffic data) [BS EN 1991-2:2003, 4.6.6 (1)]

The derivation of a site-specific model should be considered as follows:

i) where knowledge of local traffic conditions is poor;

ii) where local circumstances are very particular (e.g. sea ports).

The fatigue damaging stress cycles due to transit of recorded lorries should be assessed and counted using the rainflow counting procedure described in BS EN 1993-1-9. Fatigue damage should be assessed on the basis of stress cycles calculated from two traffic lanes only. These lanes (described as lanes 1 and 2) are the two traffic lanes that individually cause the most theoretical fatigue damage in the component under consideration.

The stress cycles obtained from analysis of recorded traffic data should be multiplied by a Dynamic Amplification Factor φ_{fat} which can be taken as $\varphi_{fat} = 1.2$ for a pavement surface of "good" roughness and $\varphi_{fat} = 1.4$ for a pavement of "medium" roughness. An additional Dynamic Amplification Factor should be applied for locations close to expansion joints as given in **4.6.1**(6) (See also Annex B of BS EN 1991-2).

The procedure for damage summation D_d should be as that given in **NA.2.26** for Fatigue Load Model 4.

NA.2.28 Collision forces on piers and other supporting members [BS EN 1991-2:2003, 4.7.2.1 (1)]

For the application of this clause, refer to BS EN 1991-1-7 and its National Annex.

NA.2.29 Collision forces on decks [BS EN 1991-2:2003, 4.7.2.2 (1) Note 1]

For the application of this clause, refer to BS EN 1991-1-7 and its National Annex.

NA.2.30 Effects of collision forces on vehicle restraint systems [BS EN 1991-2:2003, 4.7.3.3]

NA.2.30.1 [BS EN 1991-2:2003, 4.7.3.3 (1) Note 1]

The appropriate class of forces given in Table NA.6 should be selected in place of Table 4.9(n) of BS EN 1991-2, depending on specific applications.

Table NA.6 **Forces due to collision with vehicle restraint systems for determining global effects**

Class	Transverse force (kN)	Longitudinal force (kN)	Vertical force (kN)	Examples of applications
A	100	—	—	Normal containment flexible parapets (e.g. metal post and rail parapets)
B	200	—	—	Normal containment rigid parapets (e.g. reinforced concrete parapets)
C	400	100	175	Very high containment flexible parapets (e.g. metal post and rail parapets)
D	600	100	175	Very high containment rigid parapets (e.g. reinforced concrete parapets)

The forces in Table NA.6 should be applied uniformly over a length of 3 m at the top of the traffic face of the vehicle restraint system and in a position along the line of the vehicle restraint system that produces the maximum effects on the part of the structure under consideration.

NA.2.30.2 [BS EN 1991-2:2003, 4.7.3.3 (1) Note 3]

The vertical forces acting simultaneously with the collision forces should be taken as 0,75 times the loading given by Load Model 1 in **4.3.2** of BS EN 1991-2 and the full accidental wheel/vehicle loading given in **4.7.3.1** of BS EN 1991-2. The three sets of forces should be applied in a way that will have the most severe effect on the part of the structure under consideration.

NA.2.30.3 [BS EN 1991-2:2003, 4.7.3.3 (2) Note]

The recommended value should be used.

NA.2.31 Collision forces on structural members [BS EN 1991-2:2003, 4.7.3.4(1)]

Structural members above or beside the carriageway level should be provided with protective measures e.g. barriers. If not, the following options should be considered.

i) Design for vehicle collision forces; see BS EN 1991-2, **4.7.2.1** and **NA.2.28**.

ii) Design for nominal vehicle collision forces for the provision of minimum robustness and for the situation where damage or failure to the structural member will not cause collapse of the structure; see BS EN 1991-2, **4.7.3.4** (2). These nominal vehicle collision forces should be determined for the individual project. Strategies for accidental design situations are set out in BS EN 1991-1-7 and its NA.

NA.2.32 Actions on pedestrian parapets [BS EN 1991-2:2003, 4.8 (1) Note 2]

The required class of pedestrian parapet for the particular situation should be chosen in accordance with EN 1317-6 and determined for the individual project. The characteristic value of forces transferred to the structure should be taken as the design loads given in EN 1317-6 for the relevant class of pedestrian parapet.

For the design of the supporting structure the minimum horizontal load should be taken as 1,6 kN/m, corresponding to Class E, for normal situations, and 3.0 kN/m^2, corresponding to Class J, for exceptional situations where crowding can occur. The horizontal load should be applied at the top of the pedestrian parapet and should be considered to act simultaneously with the uniformly distributed vertical loads defined in **5.3.2.1** of BS EN 1991-2.

NA.2.33 Supporting structures to pedestrian parapets, which are not adequately protected against vehicle collisions [BS EN 1991-2:2003, 4.8 (3)]

The recommended value should be used.

NA.2.34 Model for vertical loads on backfill behind abutments and wing walls adjacent to bridges [BS EN 1991-2:2003, 4.9.1 (1) Note 1]

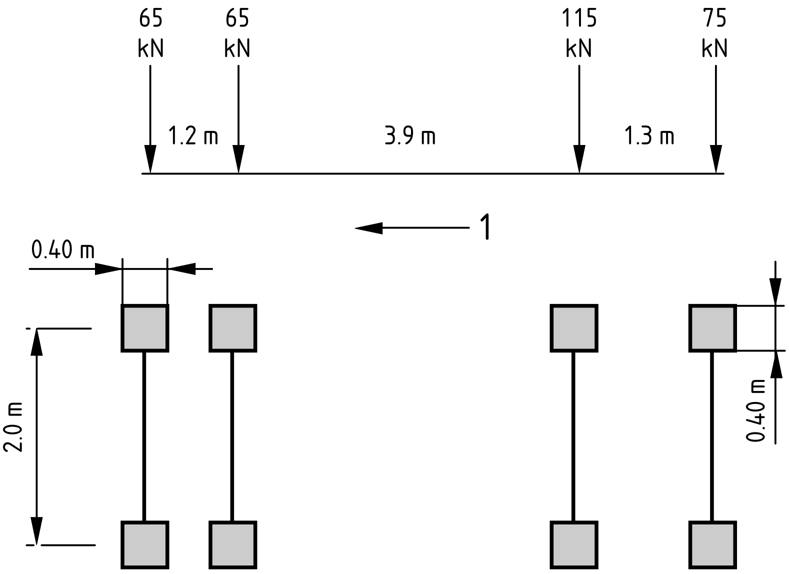
NA.2.34.1 General

For determining the vertical and horizontal pressures in the backfill behind an abutment or wing wall, the carriageway located behind the abutments is loaded with the vehicle loads as described in **NA.2.34.2** and **NA.2.34.3**. These vehicle loads should be considered as characteristic loads.

NA.2.34.2 Loading from normal traffic

The model vehicle with the configuration given in Figure NA.6 should be used. Each axle consists of two wheels of equal weight at a distance apart of 2,0 m to the centre line of each wheel. The contact surface of each wheel should be taken as a square of sides 0,40 m.

Figure NA.6 **Vehicle model for abutments and wing walls**



Key
1 = Direction of travel

All the axle loads given in Figure NA.6 should be multiplied by an Overload Factor of 1,5 and a Dynamic Amplification Factor of 1,4. The effect of the Dynamic Amplification Factor on vertical and horizontal earth pressure may be considered to reduce linearly from 1,4 at surface level to 1,0 at a depth of 7,0 m. Where appropriate, detailed modelling may be used to determine more accurately the variation of Dynamic Amplification Factor with depth. Vehicles should be positioned in a maximum of three adjacent notional lanes. The axle loads for the vehicle in the third lane should be factored by a lane factor of 0,5.

The maximum load effect from the following two cases should be used for design.

i) A single vehicle in each notional lane.

ii) Convoy of vehicles in each notional lane with the Dynamic Amplification Factor set to 1,0 (represents a traffic jam situation).

The vehicles within each lane should be positioned, laterally and longitudinally, to maximize the load effects at the part of the structure under consideration. However, a minimum lateral spacing of 1.0 m is maintained between the centrelines of wheels from two adjacent vehicles. In the case of a convoy of vehicles a minimum longitudinal spacing of 3.0 m should be kept between the last axle of the leading vehicle and the first axle of the trailing vehicle.

Where the load model behind the abutment is applied in conjunction with either Load Model 1 or Load Model 2 on the deck, the two load models should be applied simultaneously, without modification to their rules of application.

The load model should be considered to form part of gr1a and gr1b, in which it should be applied at its characteristic value, and gr2, in which it should be applied at its frequent value (see Table NA.3). Combination factors for the load model should be taken equal to those for the tandem axle system of Load Model 1.

NA.2.34.3 Loading from special vehicles

The abutments and wing walls adjacent to bridges should be designed for the effects of special vehicles (both *SV* and *SOV* models) where required.

The special vehicles for Load Model 3, given in **NA.2.16**, along with the rules of its application, should be used for this purpose. For the evaluation of vertical and horizontal pressures, due to vehicle loading behind the abutment, only one *SV* or *SOV* vehicle model, appropriate to the road class, and in one notional lane, should be considered. The vehicle load model given in **NA.2.34.2** may be applied in two adjacent lanes but with all the axle loads multiplied by a factor of 0,75.

The effect of the Dynamic Amplification Factor on vertical and horizontal earth pressure may be considered to reduce linearly from the values given in Table NA.2 at the surface to 1,0 at a depth of 7,0 m. Where appropriate, detailed modelling may be used to determine more accurately the variation of the Dynamic Amplification Factor with depth.

NA.2.35 Load models for inspection gangways [BS EN 1991-2:2003, 5.2.3 (2)]

The recommended model in BS EN 1991-2 should be used.

NA.2.36 Uniformly distributed load [BS EN 1991-2:2003, 5.3.2.1 (1)]

Where the risk of a continuous dense crowd exists (e.g. footbridges serving a sports stadium) the Load Model 4 defined in **4.3.5** of BS EN 1991-2, corresponding to q_{fk} = 5,0 kN/m^2 should be used. In other cases, the uniformly distributed load, q_{fk}, should be taken as follows.

$$q_{fk} = 2,0 + \frac{120}{L + 10}\ \text{kN/m}^2$$

$$2,5\ \text{kN/m}^2 \leq q_{fk} \leq 5,0\ \text{kN/m}^2$$

Where L is the loaded length in m.

NA.2.37 Concentrated load [BS EN 1991-2:2003, 5.3.2.2 (1)]

The characteristic value of the concentrated load Q_{fwk} given in BS EN 1991-2 should be used.

NA.2.38 Service vehicle [BS EN 1991-2:2003, 5.3.2.3 (1) Note 1]

Where footbridges do not have permanent provisions to prevent the entry of vehicles on to the footbridge, the vehicle model given in Figure 5.2 of BS EN 1991-2 with characteristic axle loads, Q_{sv1} = 115 kN and Q_{sv2} = 65 kN should be used.

NA.2.39 Horizontal force on footbridges [BS EN 1991-2:2003, 5.4 (2)]

The recommended values should be used.

NA.2.40 General actions for accidental design situations for footbridges [BS EN 1991-2:2003, 5.6.1 (1)]

No additional information is provided.

NA.2.41 Collision forces on piers of footbridges [BS EN 1991-2:2003, 5.6.2.1 (1)]

For application, refer to BS EN 1991-1-7 and its National Annex.

NA.2.42 Collision forces on decks of footbridges [BS EN 1991-2:2003, 5.6.2.2 (1)]

For application, refer to BS EN 1991-1-7 and its National Annex.

NA.2.43 Accidental presence of a heavy vehicle [BS EN 1991-2:2003, 5.6.3 (2)]

The characteristics of a vehicle, which may be accidentally present on the footbridge where no permanent obstacle is provided, is defined in **NA.2.38**. Alternative load model characteristics may be determined for the individual project.

NA.2.44 Dynamic models for pedestrian actions on footbridges [BS EN 1991-2:2003, 5.7 (3)]

NA.2.44.1 General

Dynamic models for pedestrian loads and associated comfort criteria are given below. Two distinct analyses are required:

a) the determination of the maximum vertical deck acceleration and its comparison with the comfort criteria (as described in **NA.2.44.3** to **NA.2.44.6**), and

b) an analysis to determine the likelihood of large synchronized lateral responses (as described in **NA.2.44.7**).

For unusual bridges, or in circumstances where other responses or response mechanisms are likely to cause discomfort (for example the wind buffeting of pedestrian bridges over railways), the effects of actions other than those described should be considered.

The following activities are not included and any associated requirements should be determined for the individual project:

- mass gathering (for example marathons, demonstrations);
- deliberate pedestrian synchronization;
- vandal loading.

NA.2.44.2 Dynamic actions to be considered

(1) All bridges should be categorized into bridge classes by their usage to determine the appropriate actions due to pedestrians. Group sizes for each bridge class should be applied as given in Table NA.7.

Table NA.7 **Recommended crowd densities for design**

Bridge class	**Bridge usage**	**Group size (walking)**	**Group size (jogging)**	**Crowd density** ρ **(persons/m²) (walking)**
A	Rural locations seldom used and in sparsely populated areas.	N = 2	N = 0	0
B	Suburban location likely to experience slight variations in pedestrian loading intensity on an occasional basis.	N = 4	N = 1	0.4
C	Urban routes subject to significant variation in daily usage (e.g. structures serving access to offices or schools).	N = 8	N = 2	0.8
D	Primary access to major public assembly facilities such as sports stadia or major public transportation facilities.	N = 16	N = 4	1.5

(2) Crowd loading densities to be used in design should be determined for the individual project and be appropriate for the intended bridge usage. Table NA.7 provides recommended values of crowd densities for each bridge class.

(3) Depending on the expected bridge usage, it may be determined that jogging cases given in Table NA.7 can be neglected for individual projects.

NA.2.44.3 Vertical response calculations

(1) It should be demonstrated that the peak vertical deck accelerations determined for the actions described in **NA.2.44.4** and **NA.2.44.5** are less than the limits defined in **NA.2.44.6**.

(2) In calculating the peak vertical deck accelerations account should be taken of the following.

- The load models provided should be applied in order to determine the maximum vertical acceleration at the most unfavourable location on the footbridge deck.
- The calculated vertical responses should include the effect of torsional or other motions.
- Modes other than the fundamental mode may need to be taken into account in order to calculate the maximum responses.

- When the vertical deck modes are not well separated, consideration should be given to the use of more sophisticated methods of analysis, in order to determine combined mode responses. In all cases, it is conservative to use the vector sum of the peak accelerations for those modes that need such combination.

NA.2.44.4 Dynamic actions representing the passage of single pedestrians and pedestrian groups

(1) The design maximum vertical accelerations that result from single pedestrians or pedestrian groups should be calculated by assuming that these are represented by the application of a vertical pulsating force F (N), moving across the span of the bridge at a constant speed v_t, as follows:

$$F = F_0 . k(f_v) . \sqrt{1 + \gamma .(N - 1)} . \sin(2\pi . f_v . t)$$

Where:

N is the number of pedestrians in the group obtained from **NA.2.44.2**.

F_0 is the reference amplitude of the applied fluctuating force (N) given in Table NA.8 (and represents the maximum amplitude of the applied pedestrian force at the most likely pace frequency).

f_v is the natural frequency (Hz) of the vertical mode under consideration.

$k(f_v)$ given in Figure NA.8, is a combined factor to deal with (a) the effects of a more realistic pedestrian population, (b) harmonic responses and (c) relative weighting of pedestrian sensitivity to response.

t elapsed time (seconds).

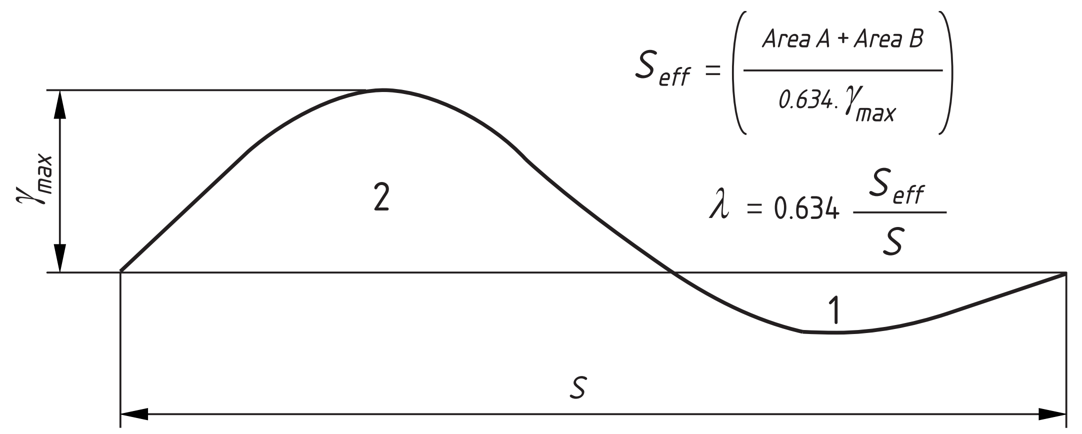
γ is a reduction factor to allow for the unsynchronized combination of actions in a pedestrian group, is a function of damping and effective span, and is obtained from Figure NA.9.

S_{eff} is an effective span length (m) equal to the area enclosed by the vertical component of the mode shape of interest divided by 0.634 times the maximum of the vertical component of the same mode shape (see Figure NA.7).

(In all cases it is conservative to use $S_{eff} = S$).

S is the span of the bridge (m).

Figure NA.7 **Effective span calculation**



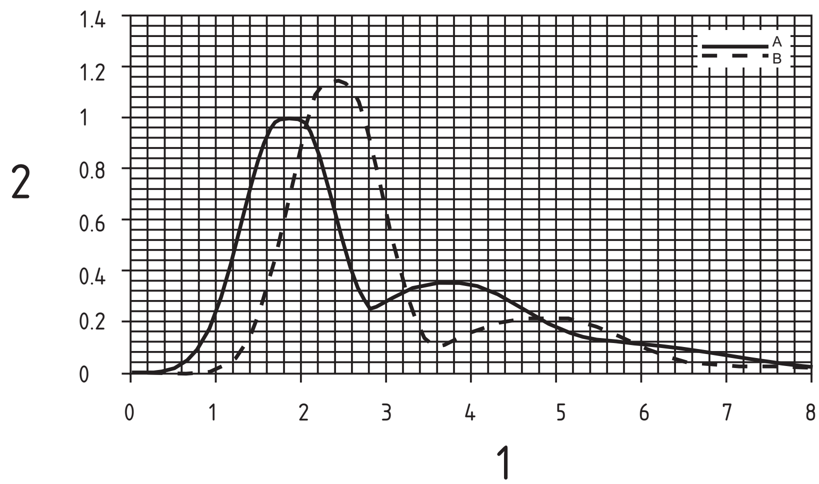
Key

1 = Area B   2 = Area A

Table NA.8 **Parameters to be used in the calculation of pedestrian response**

Load parameters	**Walking**	**Jogging**
Reference load, F_0 (N)	280	910
Pedestrian crossing speed, v_t (m/sec)	1,7	3

Figure NA.8 **Relationships between $k(f_v)$ and mode frequencies f_v**



Key

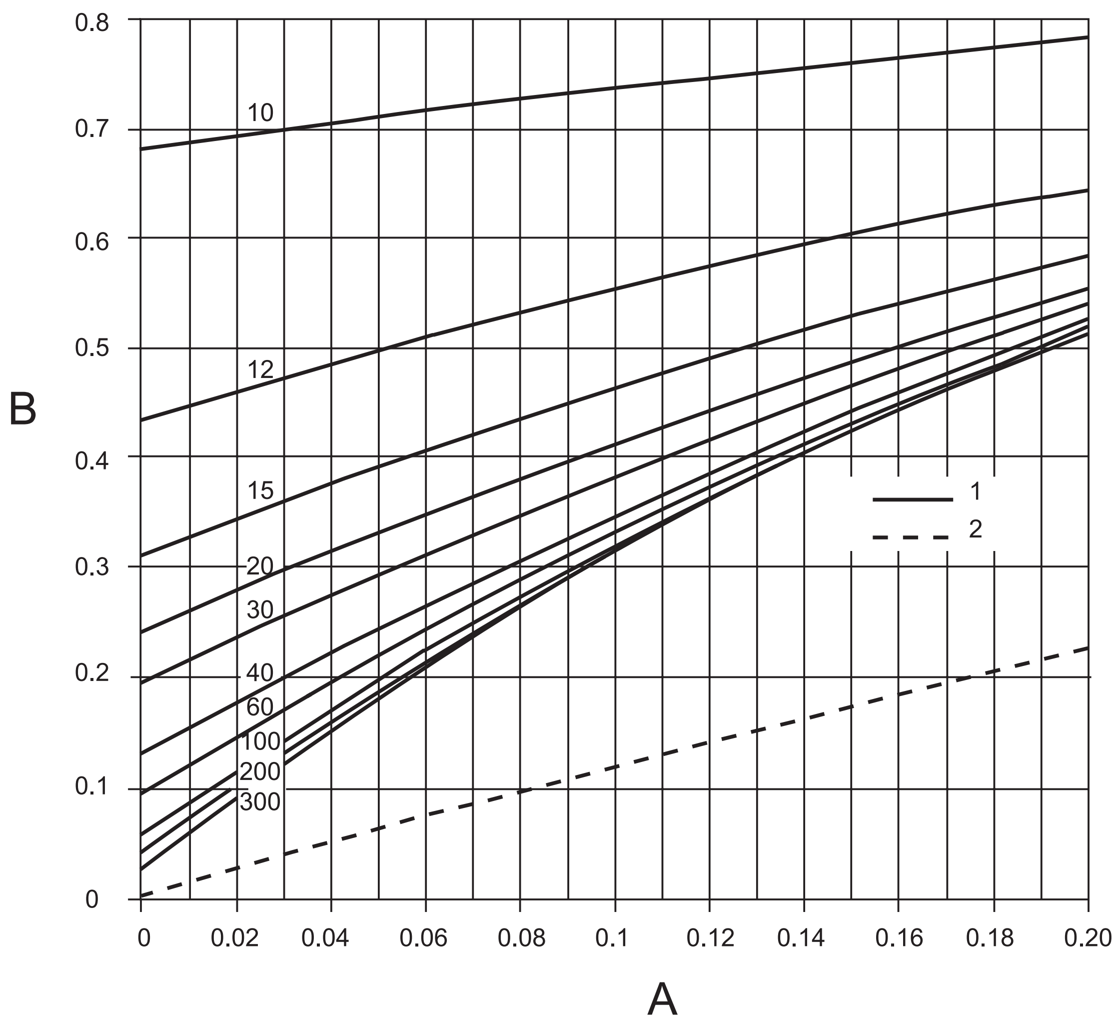
A = Walking

B = Jogging/running

1 = Mode frequency f_v (Hz)

2 = Combined population and harmonic factor *k(fv)*

Figure NA.9 **Reduction factor, γ, to allow for the unsynchronized combination of pedestrian actions within groups and crowds**



Key

1 = Pedestrian groups

2 = Crowd loading

A = Structural damping – logarithmic decrement, δ

B = Reduction factor on effective number of pedestrians, γ

NOTE All curves represent the variation of the reduction factor with structural damping for the value of effective span, S_{eff} (m), given

NA.2.44.5 Steady state modelling of pedestrians in crowded conditions

(1) The design maximum vertical accelerations that result from pedestrians in crowded conditions may be calculated by assuming that these are represented by a vertical pulsating distributed load w (N/m^2), applied to the deck for a sufficient time so that steady state conditions are achieved as follows:

$$w = 1.8\left(\frac{F_0}{A}\right).k(f_v).\sqrt{\gamma.N/\lambda}.\sin(2\pi.f_v.t)$$

Where:

N is the total number of pedestrians distributed over the span S.

$N = \rho A = \rho S b$

ρ is the required crowd density obtained from **NA.2.44.2** but with a maximum value of 1.0 persons/m^2. (This is because crowd densities greater than this value produce less vertical response as the forward motion slows.)

S is the span of the bridge (m)

b is the width of the bridge subject to pedestrian loading

γ is a factor to allow for the unsynchronized combination of actions in a crowd and is obtained from Figure NA.9.

λ is a factor that reduces the effective number of pedestrians when loading from only part of the span contributes to the mode of interest. $\lambda = 0.634(S_{eff} / S)$.

For other symbols see **NA.2.44.4** (1).

(2) In order to obtain the most unfavourable effect this loading should be applied over all relevant areas of the footbridge deck with the direction of the force varied to match the direction of the vertical displacements of the mode for which responses are being calculated.

(3) Understanding of the dynamic response of structures in crowded conditions is still evolving and there is evidence to suggest that the peak acceleration arising from the application of w as specified in **NA.2.44.5** (1) may be conservative in some cases. Alternatively appropriate dynamic models may be determined for the individual project.

NA.2.44.6 Recommended serviceability limits for use in design

(1) The maximum vertical acceleration calculated from the above actions should be less than the design acceleration limit given by:

$a_{\text{limit}} = 1.0\ k_1\ k_2\ k_3\ k_4$ m/s^2

and 0.5 m/s$^2 \leq a_{\text{limit}} \leq$ 2,0 m/s^2

Where:

k_1, k_2 and k_3 are the response modifiers taken from Tables NA.9 to NA.11 in which:

k_1 = site usage factor

k_2 = route redundancy factor

k_3 = height of structure factor.

k_4 is an exposure factor which is to be taken as 1.0 unless determined otherwise for the individual project. See also **NA.2.44.6** (2).

Table NA.9 **Recommended values for the site usage factor k_1**

Bridge function	k_1
Primary route for hospitals or other high sensitivity routes	0,6
Primary route for school	0.8
Primary routes for sports stadia or other high usage routes	0.8
Major urban centres	1,0
Suburban crossings	1,3
Rural environments	1,6

Table NA.10 **Recommended values for the route redundancy factor k_2**

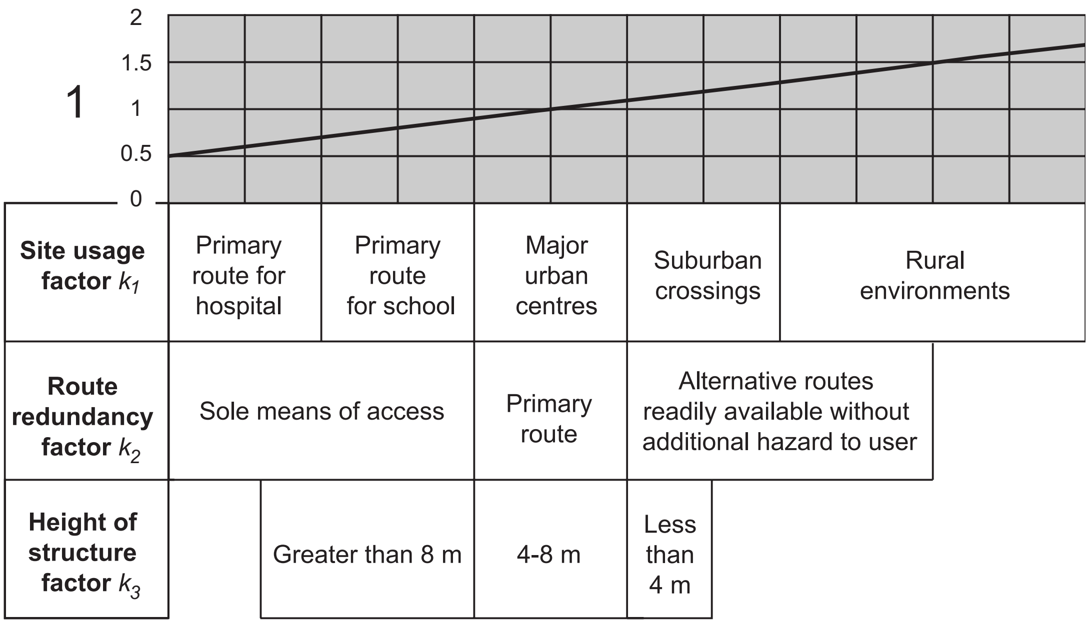
Route redundancy	k_2
Sole means of access	0,7
Primary route	1,0
Alternative routes readily available	1,3

Table NA.11 **Recommended values for the structure height factor k_3**

Bridge height	k_3
Greater than 8 m	0,7
4 m to 8 m	1,0
Less than 4 m	1,1

Values of k_1, k_2 and k_3 other than those given in Tables NA.9 to NA.11 may be determined for the individual project using Figure NA.10 as a guide.

Figure NA.10 **Response modifiers**



Key

1 = Response modifier, k_i

(2) k_4 may be assigned a value of between 0.8 and 1.2 to reflect other conditions that may affect the users' perception towards vibration. These may include consideration of parapet design (such as height, solidity or opacity), quality of the walking surface (such as solidity or opacity) and provision of other comfort-enhancing features. The value to be taken should be determined for the individual project.

(3) For some types of bridges (for example bridges in remote locations), less onerous design limits may be applied, where a suitable risk assessment has been carried out. Any relaxation of the design limits should be determined for the individual project.

NA.2.44.7 The avoidance of unstable lateral responses due to crowd loading

(1) Structures should be designed to avoid unintended unstable lateral responses.

(2) If there are no significant lateral modes with frequencies below 1.5 Hz it may be assumed that unstable lateral responses will not occur.

(3) For all other situations, it should be demonstrated that unstable lateral responses due to crowd loading will not occur, using the following method.

For all deck modes of vibration having a significant lateral horizontal component and a frequency below 1.5 Hz, compare the pedestrian mass damping parameter, D, and the mode frequency with the stability boundary defined in Figure NA.11. If the pedestrian mass damping parameter falls below the indicated boundary divergent lateral responses may be expected. Values above the line should be stable.

The pedestrian mass damping parameter D is given by:

$$D = \frac{m_{bridge} \cdot \xi}{m_{pedestrian}}$$

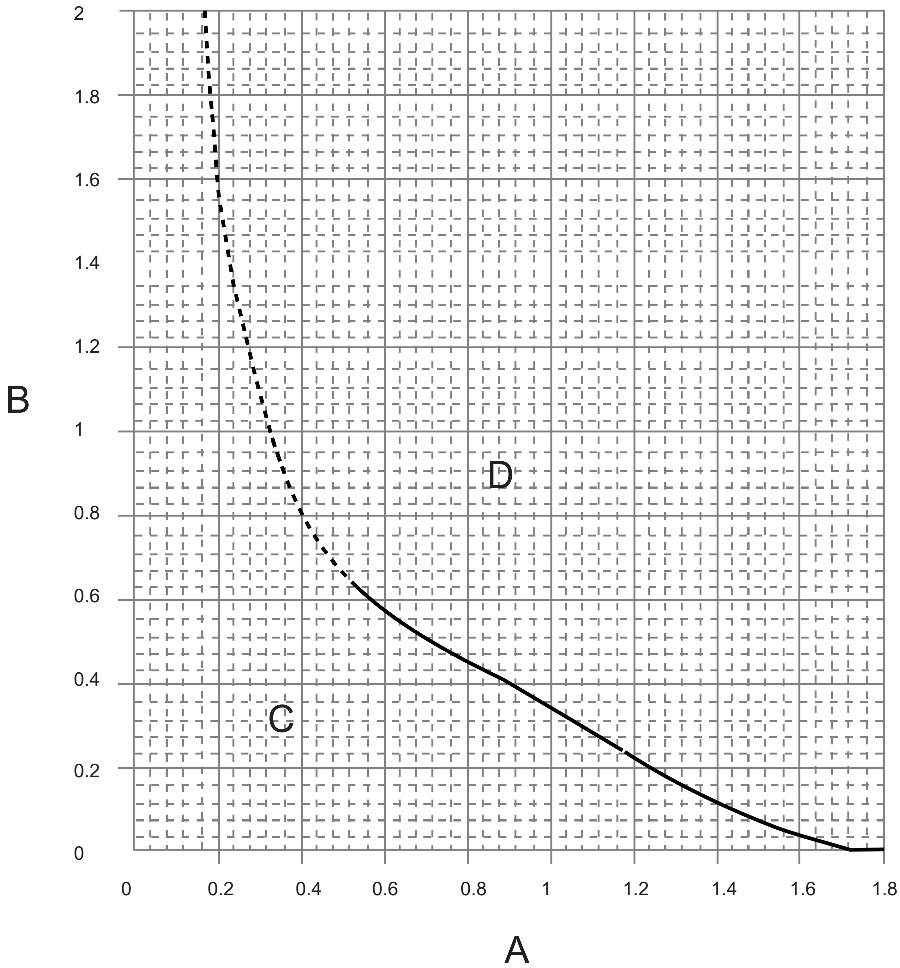
m_{bridge} is the mass per unit length of the bridge

$m_{pedestrian}$ is the mass per unit length of pedestrians for the relevant crowd density obtained from **NA.2.44.2** assuming that each pedestrian weighs 70 kg

ξ is the structural damping when expressed as a damping ratio, $\xi = \delta/(2\pi)$

δ logarithmic decrement of decay of vibration between successive peaks

Figure NA.11 **Lateral lock-in stability boundaries**



Key

A = Frequency of lateral mode (Hz)

B = Pedestrian mass damping parameter, *D*

C = Unstable

D = Stable

NOTE Reliable test measurements are only available for footbridge lateral frequencies in the range of 0.5 to 1.1 Hz. The extensions to the stability curve beyond this region are based upon a theoretical model of response only and should be used with caution.

NA.2.45 Alternative load models for railway bridges [BS EN 1991-2:2003, 6.1 (2)]

Alternative load models for non-public footpaths and actions due to traction and braking should be as set out in the following.

NA.2.45.1 Actions for non-public footpaths

The values recommended in **6.3.7** of BS EN 1991-2 should be used except as follows.

6.3.7 (2) In addition, where the walkway supports a cable route, an allowance of 1 kN/m or the actual weight of the cables, whichever is greater.

6.3.7 (3) For the design of local elements a concentrated load Q_k = 2,0 kN applied to a circular area of 100 mm diameter, or a concentrated load of 1 kN, whichever has the more severe effect.

6.3.7 (4) Horizontal handrail loading of 0.74 kN/m or a horizontal force of 0.5 kN applied at any point to the top rail, whichever has the more severe effect.

NA.2.45.2 Actions due to traction and braking

Actions due to traction and braking should be taken as the greater of equations 6.20 and 6.21, or the following.

i) Provision should be made for the nominal loads due to traction and application of brakes as given in Table NA.12. These loads are considered as acting at rail level in a direction parallel to the tracks. No addition for dynamic effects should be made to the longitudinal loads calculated as specified in this subclause.

ii) For bridges supporting ballasted track, up to one-third of the longitudinal loads may be assumed to be resisted by track outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on, or within, 18 m of either end of the bridge.

iii) Structures and elements carrying single tracks should be designed to carry the larger of the two loads produced by traction and braking in either direction parallel to the track.

iv) Where a structure or an element carries two tracks, both tracks are considered as being occupied simultaneously. Where the tracks carry traffic in opposite directions, the load due to braking should be applied to one track and the load due to traction to the other. Structures and elements carrying two tracks in the same direction should be subjected to braking or traction on both tracks, whichever gives the greater effect. Consideration should be given to braking and traction, acting in opposite directions, producing rotational effects.

v) Where elements carry more than two tracks, longitudinal loads should be considered as applied simultaneously to two tracks only.

Table NA.12 **Nominal longitudinal loads**

Standard loading type	**Load arising from**	**Loaded length (m)**	**Longitudinal load (kN)**
Load Model 71, SW/0 and HSLM	Traction (30% of load on driving wheels)	up to 3	150
		from 3 to 5	225
		from 5 to 7	300
		from 7 to 25	24 (L – 7) + 300
		over 25	750
	Braking (25% of load on braked wheels)	up to 3	125
		from 3 to 5	187
		from 5 to 7	250
		over 7	20 (L – 7) + 250

NA.2.46 Other types of railways [BS EN 1991-2:2003, 6.1 (3)P]

The loading and characteristic values of actions should be determined for the individual project (for example for light rail systems and underground railways).

NA.2.47 Temporary bridges [BS EN 1991-2:2003, 6.1 (7)]

The requirements for temporary railway bridges should be determined for the individual project.

NA.2.48 Values of α factor [BS EN 1991-2:2003, 6.3.2 (3)P]

The value of α should be taken as 1.1.

Alternative values of α may be determined for the individual project.

NA.2.49 Choice of lines for heavy rail traffic [BS EN 1991-2:2003, 6.3.3 (4)P]

Generally there is no requirement to design for SW/2 loading in the UK.

Alternative requirements for heavy rail traffic may be determined for the individual project.

NA.2.50 Alternative requirements for a dynamic analysis [BS EN 1991-2:2003, 6.4.4 (1)]

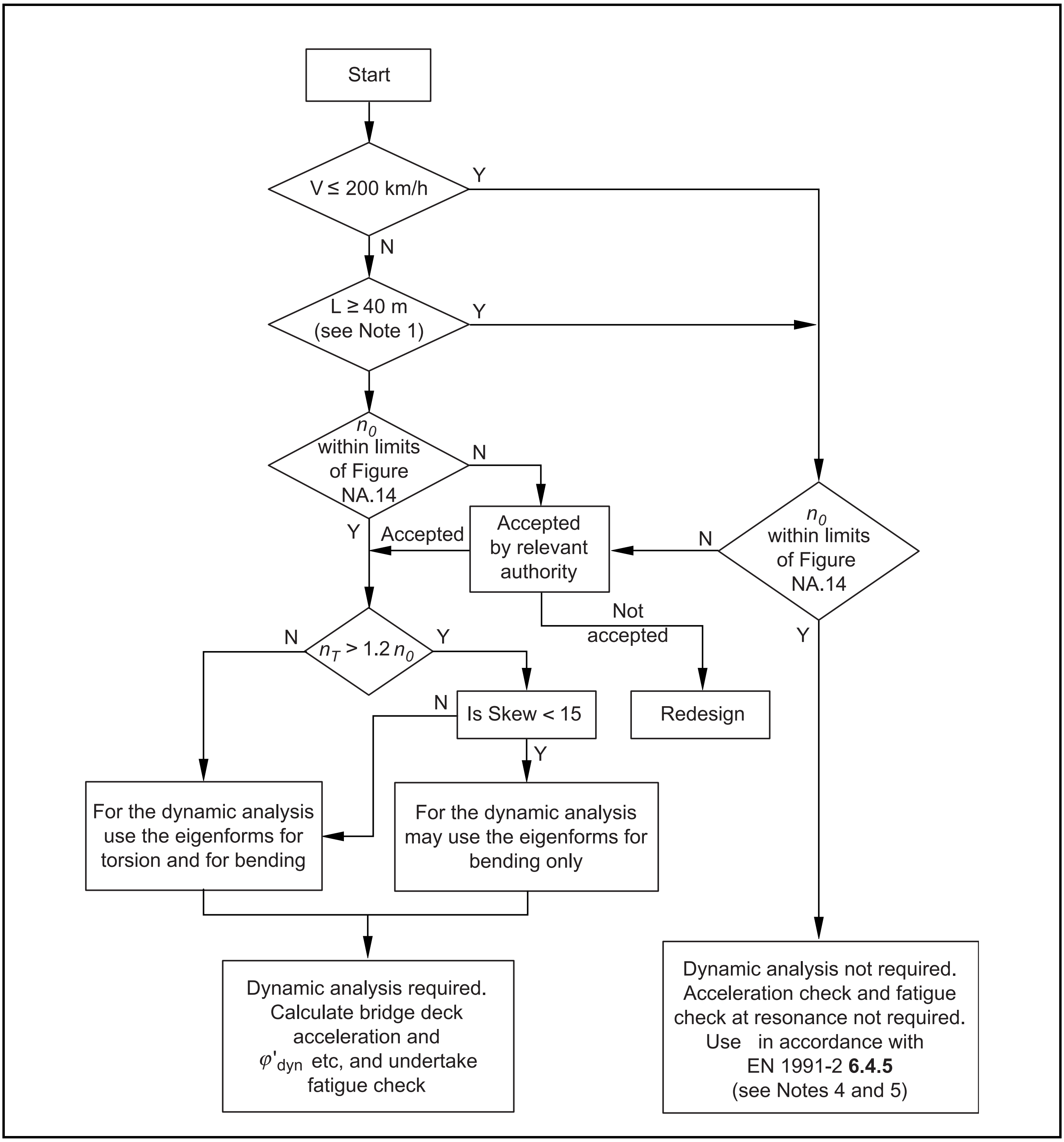
The requirements for determining whether a dynamic analysis is required (in addition to static analysis) are shown in Figures NA.12 and NA.13. Figure NA.12 is only applicable to simple structures that exhibit only longitudinal line beam behaviour. Figure NA.13 is applicable to both simple and complex structures.

NOTE 1 When determining whether a dynamic analysis is required it is essential to differentiate between simple and complex structures, i.e. those which exhibit only longitudinal line beam behaviour and may be represented by line beams, and those that exhibit longitudinal/transverse behaviour which require more complex representation/modelling.

NOTE 2 Simple structures which exhibit longitudinal line beam behaviour with insignificant contributions from other dynamic modes will generally comprise of deck type structures of slab, beam and slab or box and slab construction where the tracks are located over the webs of longitudinal spanning elements and where the deck/floor elements are not required to directly distribute axle/wheel load effects to the longitudinal elements by transverse bending.

NOTE 3 Complex structures require deck/floor elements to distribute axle/wheel loads to primary longitudinal elements. Complex structures will typically include through/half through structures with primary transverse spanning deck/floors, as well as deck type structures of beam and slab (or box and slab) construction where the deck/floor elements are required to distribute loads to the longitudinal elements in bending.

Figure NA.12 **Flow chart for determining whether a dynamic analysis is necessary for "simple" structures**



Where:

V is the maximum line speed at the site (km/h)

L is the span length (m)

n_0 is the first natural bending frequency of the bridge loaded by permanent actions (Hz)

n_T is the first natural torsional frequency of the bridge loaded by permanent actions (Hz)

NOTE 1 Simply supported structure only with negligible skew and rigid supports.

NOTE 2 For bridges with a first natural frequency within the limits given by Figure NA.9 and a maximum line speed at the site V_{line} not exceeding 200 km/h a dynamic analysis is not required.

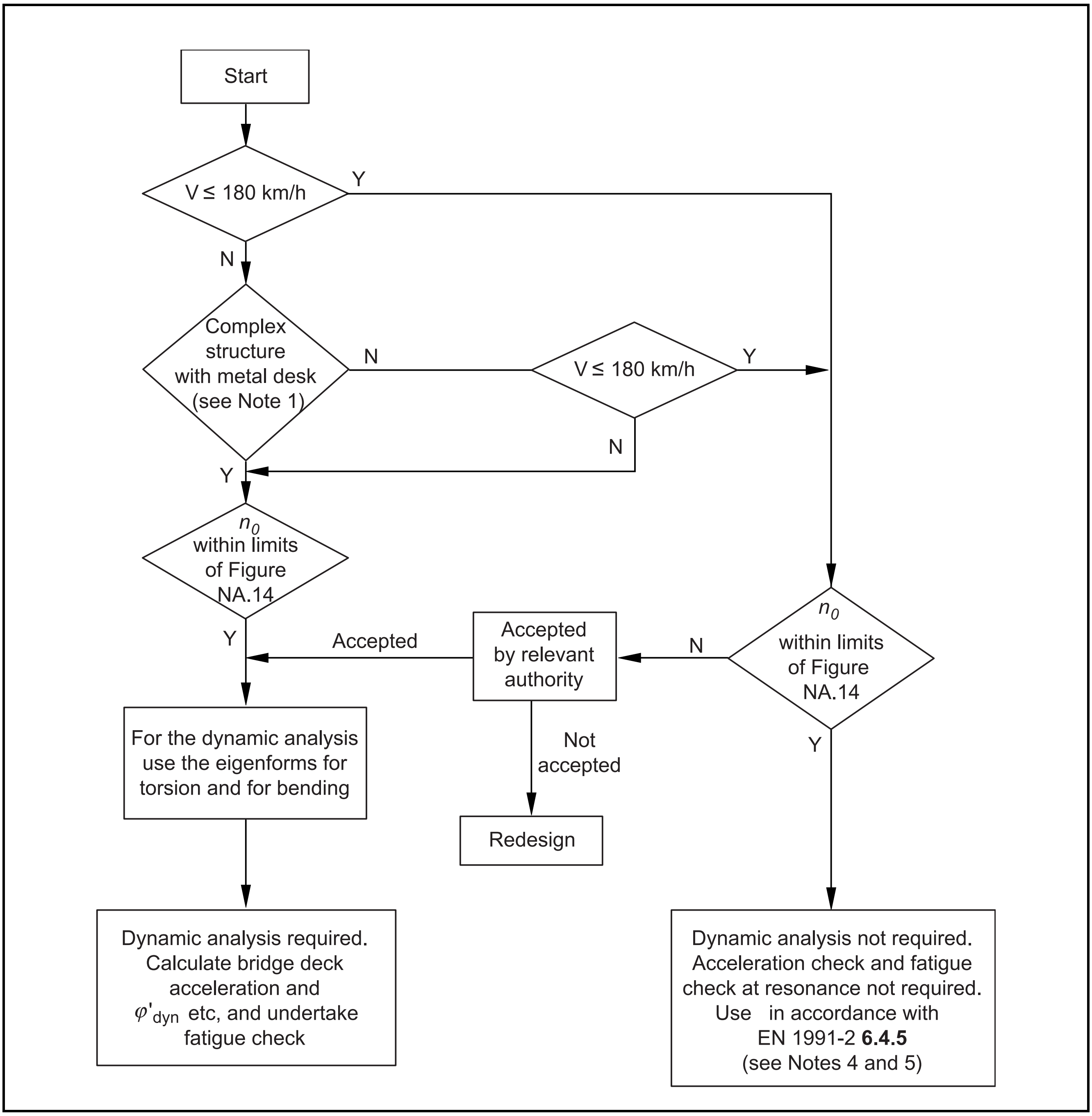
*NOTE 3 φ'_{dyn} is the dynamic impact increment for Real Trains or Load Model HSLM for the structure given in **6.4.6.5** (3).*

*NOTE 4 A dynamic analysis is required where the Frequent Operating speed of a Real Train equals a Resonant Speed of the structure [see BS EN 1991-2 **6.4.6.6** (2)].*

*NOTE 5 Valid providing the bridge meets the requirements for resistance, deformation limits given in Annex A2 to BS EN 1990, **A2.4.4** and the maximum coach body acceleration (or associated deflection limits) corresponding to a very good standard of passenger comfort given in Annex A2 to BS EN 1990, **A2.4.4.3**.*

NOTE 6 This figure is only applicable to structures which may be represented by line beams

Figure NA.13 **Flow chart for determining whether a dynamic analysis is required for "simple" and "complex" structures**



Where:

V is the maximum line speed at the site (km/h)

L is the span length (m)

n_0 is the first natural bending frequency of the bridge loaded by permanent actions (Hz)

n_T is the first natural torsional frequency of the bridge loaded by permanent actions (Hz)

NOTE 1 Metallic floors with closely spaced transverse 'T' ribs (e.g. as utilized in "Western Region Box Girder Structures") may be assumed to have an adequate dynamic response for speeds up to 200 km/h when designed with the following characteristics: a minimum deck plate thickness of 30 mm, maximum spacing of transverse 'T' ribs not greater than 610 mm and satisfying minimum fatigue design requirements of 18-27 million tonnes of heavy traffic per annum.

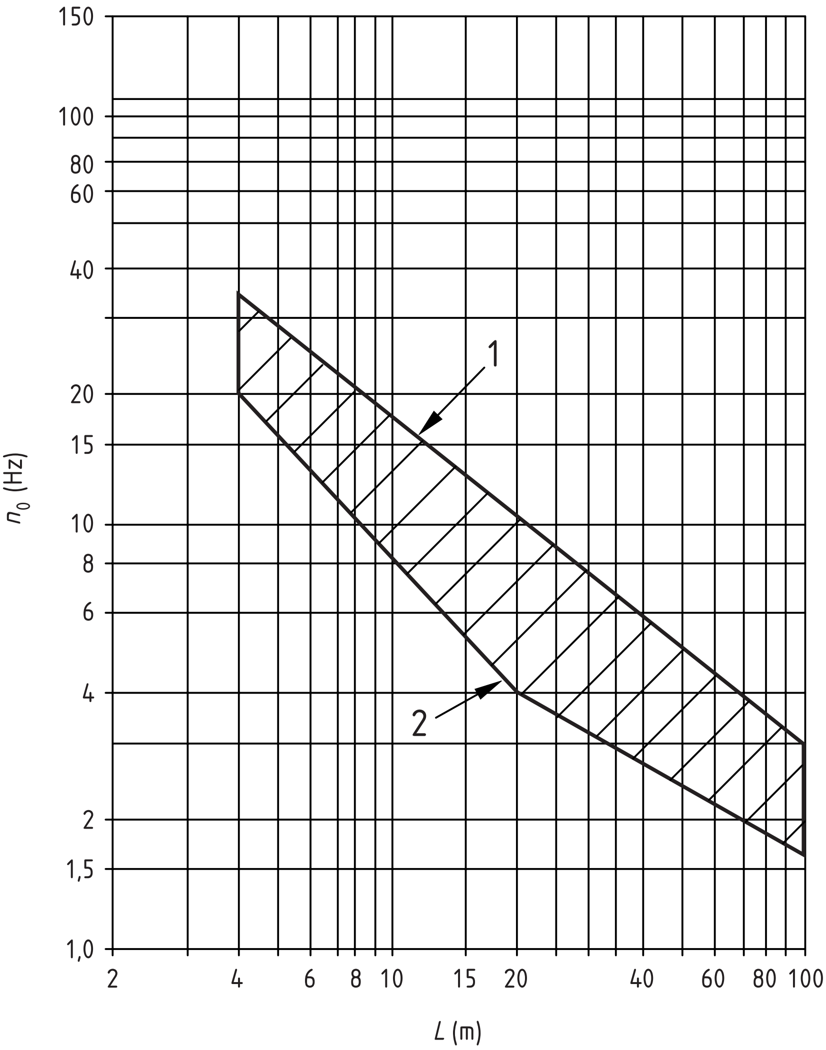
NOTE 2 For bridges with a first natural frequency within the limits given by Figure NA.14 and a Maximum Line speed at the Site V_{line} not exceeding 200 km/h a dynamic analysis is not required.

NOTE 3 φ'_{dyn} is the dynamic impact increment for Real Trains or Load Model HSLM for the structure given in ***6.4.6.5*** *(3).*

NOTE 4 A dynamic analysis is required where the Frequent Operating Speed of a Real Train equals a Resonant Speed of the structure [see BS EN 1991-2, ***6.4.6.6****(2)].*

NOTE 5 Valid providing the bridge meets the requirements for resistance, deformation limits given in Annex A2 to BS EN 1990, ***A2.4.4****, and the maximum coach body acceleration (or associated deflection limits) corresponding to a very good standard of passenger comfort given in Annex A2 to BS EN 1990,* ***A2.4.4.3****.*

Figure NA.14 **Limits of bridge natural frequency n_0 in Hz as a function of L in m**



Key
1 = Upper limit of natural frequency
2 = Lower limit of natural frequency

Where:

n_0 is the first natural vertical mode bending frequency of the unloaded bridge [i.e. permanent (including removable) loads only]

L is the span length for simply supported bridges or L_Φ for other bridge types.

The upper limits of n_0 is governed by the limits of application of the allowances for the dynamic increments due to track irregularities and is given by:

$n_{0\,=}\ 94.76 \times L^{-0.748}$

The lower limit of n_0 is governed by dynamic impact criteria and is given by:

$n_0 = 80/L$ for 4 m $\leq L \leq$ 20 m

$n_0 = 23.58 \times L^{-0.592}$ for 20 m $\leq L \leq$ 100 m.

NA.2.51 Choice of dynamic factor [BS EN 1991-2:2003, 6.4.5.2 (3)P]

Generally Φ_3 should be used.

Alternative values may be determined for the individual project.

NA.2.52 Alternative values of determinant length [BS EN 1991-2:2003, 6.4.5.3 (1) Table 6.2]

The values of determinant length given in Table 6.2 should be used with the following modifications.

Case 1.1 – Deck plate (for both directions) – the lesser of three times cross girder spacing or cross girder spacing + 3 m.

Case 2.1 – Deck plate (for both directions) – cross girder spacing + 3 m.

Case 5.7 – Longitudinal cantilevers.

NA.2.53 Determinant length of transverse cantilevers [BS EN 1991-2:2003, 6.4.5.3, Note a, Table 6.2]

The loading to be used for establishing the determinant length of transverse cantilevers should be determined for the individual project.

NA.2.54 Additional requirements for the application of HSLM [BS EN 1991-2:2003, 6.4.6.1.1 (6), Table 6.4]

Additional requirements for the application of HSLM-A and HSLM-B may be determined for the individual project.

NA.2.55 Loading and methodology for dynamic analysis [BS EN 1991-2:2003, 6.4.6.1.1 (7)]

The loading and methodology for the analysis should be determined for the individual project.

NA.2.56 Additional load cases depending upon number of tracks [BS EN 1991-2:2003, 6.4.6.1.2 (3), Table 6.5, Note a)]

The loading should be determined for the individual project.

NA.2.57 Values of damping [BS EN 1991-2:2003, 6.4.6.3.1 (3) Table 6.6]

Alternative values for damping should be determined for the individual project.

NA.2.58 Alternative density values of materials [BS EN 1991-2:2003, 6.4.6.3.2 (3)]

Alternative density values may be determined for the individual project.

NA.2.59 Enhanced Young's modulus [BS EN 1991-2:2003, 6.4.6.3.3 (3) Note 1]

Alternative Young's modulus values may be determined for the individual project.

NA.2.60 Other material properties [BS EN 1991-2:2003, 6.4.6.3.3 (3) Note 2]

Other material properties may be determined for the individual project.

NA.2.61 Reduction of peak response at resonance taking account of additional damping due to vehicle/bridge interaction [BS EN 1991-2:2003, 6.4.6.4 (4)]

The method should be determined for the individual project.

NA.2.62 Allowance for track defects and vehicle imperfections [BS EN 1991-2:2003, 6.4.6.4 (5)]

Generally $(1 + \varphi'')$ should be used for line speeds less than 160 km/h and $(1 + \varphi''/2)$ should be used for line speeds of 160 km/h and above.

Alternative requirements may be determined for the individual project.

NA.2.63 Increased height of centre of gravity for centrifugal forces [BS EN 1991-2:2003, 6.5.1 (2)]

Generally centrifugal forces should be taken to act outwards in a horizontal direction at a height of 1,80 m above the running surface.

Alternative values may be determined for the individual project.

NA.2.64 Actions due to braking for loaded lengths greater than 300 m [BS EN 1991-2:2003, 6.5.3 (5)]

Additional requirements may be determined for the individual project.

NA.2.65 Alternative requirements for the application of traction and braking forces [BS EN 1991-2:2003, 6.5.3 (9)]

The requirements of **6.5.3** (9) apply.

NA.2.66 Combined response of structure and track, requirements for non-ballasted track [BS EN 1991-2:2003, 6.5.4.1 (5)]

The requirements for non-ballasted track should be determined for the individual project.

NA.2.67 Alternative requirements for temperature range [BS EN 1991-2:2003, 6.5.4.3 (2) Notes 1 and 2]

The requirements of **6.5.4.3** (2) apply.

NA.2.68 Longitudinal shear resistance between track and bridge deck [BS EN 1991-2:2003, 6.5.4.4 (2) Note 1]

The values should be determined for the individual project.

NA.2.69 Alternative design criteria [BS EN 1991-2:2003, 6.5.4.5]

Alternative requirements may be determined for the individual project.

NA.2.70 Minimum value of track radius [BS EN 1991-2:2003, 6.5.4.5.1 (2)]

Alternative requirements should be determined for the specific project.

NA.2.71 Alternative calculation methods [BS EN 1991-2:2003, 6.5.4.6]

Alternative calculation methods may be determined for the individual project.

NA.2.72 Alternative criteria for simplified calculation methods [BS EN 1991-2:2003, 6.5.4.6.1 (1)]

Alternative criteria may be determined for the individual project.

NA.2.73 Longitudinal plastic shear resistance between track and bridge deck [BS EN 1991-2:2003, 6.5.4.6.1 (4)]

Alternative values of k may be determined for the individual project.

NA.2.74 Aerodynamic actions, alternative values [BS EN 1991-2:2003, 6.6.1 (3)]

The aerodynamic actions due to static pressure changes as a train passes a structure should be determined for the individual project.

NA.2.75 Derailment of rail traffic, additional requirements [BS EN 1991-2:2003, 6.7.1 (2)P]

Alternative loading requirements for the design of railway structures to resist derailment actions from rail traffic, are appropriate for the design of all deck plates and similar local elements. These elements should be designed to support a concentrated load of $\alpha \times 1.4 \times 250$ kN (where α has a minimum value of 1,0) applied anywhere on the deck plate or local element. No dynamic factor needs to be applied to this design load.

NA.2.76 Derailment of rail traffic, measures for structural elements situated above the level of the rails and requirements to retain a derailed train on the structure [BS EN 1991-2:2003, 6.7.1 (8)P Note 1]

Measures to mitigate the consequences of a derailment may be determined for the individual project.

NA.2.77 Other actions [BS EN 1991-2:2003, 6.7.3 (1)P Note]

The requirements for other actions, including for any accidental design situation, may be determined for the individual project.

NA.2.78 Number of tracks loaded when checking drainage and structural clearances [BS EN 1991-2:2003, 6.8.1 (11)P Table 6.10]

Structural clearance requirements should be checked with rail traffic actions corresponding to the number of tracks loaded in accordance with the requirements for the number of tracks to be loaded in Table 6.10 for "Traffic Safety Checks: Vertical deformation of the deck".

Deformation due to railway traffic may be neglected when checking drainage requirements.

NA.2.79 Assessment of groups of loads [BS EN 1991-2:2003, 6.8.2 (2)]

The factors given in Table 6.11 should be used.

Where economy is not adversely affected, the recommended factors may be increased to 1.0 to simplify the design process.

NA.2.80 Frequent values of multi-component actions [BS EN 1991-2:2003, 6.8.3.1 (1)]

The factors given in Table 6.11 should be used.

Where economy is not adversely affected, the recommended factors may be increased to 1.0 to simplify the design process.

NA.2.81 Quasi-permanent values of multi-component actions [BS EN 1991-2:2003, 6.8.3.2 (1)]

The value given in **6.8.3.2** (1) should be used.

NA.2.82 Fatigue load models, structural life [BS EN 1991-2:2003, 6.9 (6)]

The design working life should be taken as 120 years.

NA.2.83 Fatigue load models, specific traffic [BS EN 1991-2:2003, 6.9 (7)]

A special traffic mix may be determined for the individual project.

NA.2.84 Dynamic factor [BS EN 1991-2:2003, Annex C (3)P]

Generally C.1 should be used for line speeds less than 160 km/h and C.2 should be used for line speeds of 160 km/h and above.

Alternative requirements may be specified for the individual project and agreed with the relevant authority.

NA.2.85 Method of dynamic analysis [BS EN 1991-2:2003, Annex C (3)P]

The method to be used should be determined for the individual project.

NA.2.86 Partial safety factor for fatigue loading [BS EN 1991-2:2003, Annex D2 (2)]

The recommended value of γ_{Ff} = 1,00 should be used.

NA.3 Decision on the status of informative annexes

NA.3.1 Load Model 3: Models of special vehicles [BS EN 1991-2:2003, Annex A]

Load Model 3 should not be used. The models for special vehicles should be those given in **NA.2.16**.

NA.3.2 Fatigue life assessment for road bridges: Assessment method based on recorded traffic [BS EN 1991-2:2003, Annex B]

The Annex B may be used in conjunction with the requirements of **NA.2.27** for Fatigue Load Model 5.

NA.3.3 Limits of validity of Load Model HSLM [BS EN 1991-2:2003, Annex E]

E1 May be used.

E2 Should not be used unless suitability of the methodology is determined for the individual project.

NOTE The methodology is not suitable for many UK structural configurations.

NA.3.4 Criteria to be satisfied if a dynamic analysis is not required [BS EN 1991-2:2003, Annex F]

Should not be used.

NOTE The loading used to derive the criteria in Annex F has been superseded by requirements in the HS INS TSI relating to load model HSLM. Additionally the methodology is not suitable for most UK structural configurations.

NA.3.5 Combined responses [BS EN 1991-2:2003, Annex G]

May be used.

NA.3.6 Load Models for rail traffic loads [BS EN 1991-2:2003, Annex H]

May be used.

NA.4 References to non-contradictory complementary information

The following is a list of references that contain non-contradictory complementary information for use with BS EN 1991-2:2003.

BS EN 1317-2, *Road restraint systems – Part 2: Performance classes, impact test acceptance criteria and test methods for safety barriers*

PD 6688-2, *Guidance for the design of structures to BS EN 1991-2*

TD 19 (DMRB 2.2.8) *Requirements for Road Restraint Systems, Highways Agency Design Manual for Roads and Bridges*

Bibliography

Standards publication

For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS EN 1991-1-7, *Eurocode 1: Actions on structures – Part 1-7: General actions – Accidental actions*

PD 6688-2, *Guidance for the design of structures to BS EN 1991-2*

Other publications

prEN 1317-6, *Road Restraint Systems – Part 6: Pedestrian restraint systems. Pedestrian parapet*

TD 19 (DMRB 2.2.8) *Requirements for Road Restraint Systems. Highways Agency Design Manual for Roads and Bridges*

UIC 777-2R, *Structures built over railway lines – Construction requirements in the track zone*

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