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Background to the National Annex to BS EN 1991-2

Traffic loads on bridges

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Foreword

Publishing information

This Published Document is published by BSI and came into effect on 31 March 2011. It was prepared by Subcommittee B/525/10, *Bridges*, under the authority of Technical Committee B/525, *Building and civil engineering*. A list of organizations represented on this committee can be obtained on request to its secretary.

Relationship with other publications

This Published Document is a background paper that gives non-contradictory complimentary information for use in the UK with the Eurocode for actions on structures, BS EN 1991-2 and its UK National Annex.

BS 5400-2 has been withdrawn and superseded by BS EN 1990, BS EN 1991-2 and BS EN 1991-1-7. References to BS 5400-2 in this Published Document are informative only and are included to provide readers with a comparison between the withdrawn British Standard and the current Eurocodes.

The following standards BS EN 1991-1-3, BS EN 1991-1-4, BS EN 1991-1-5 and BS EN 1991-1-6 should be consulted when designing bridges.

Presentational conventions

The word "should" is used to express recommendations of this Published Document. The word "may" is used in the text to express permissibility, e.g. as an alternative to the primary recommendation of the clause. The word "can" is used to express possibility, e.g. a consequence of an action or an event.

Notes and commentaries are provided throughout the text of this standard. Notes give references and additional information that are important but do not form part of the recommendations. Commentaries give background information.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a Published Document cannot confer immunity from legal obligations.

1 Scope

This Published Document gives non-contradictory, complementary information on traffic loads on bridges for use in the UK in conjunction with BS EN 1991-2 and its UK National Annex.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS EN 1990:2002, *Eurocode – Basis of structural design*

BS EN 1991-2:2003, *Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges*

NA to BS EN 1990:2002+A1:2005, *UK National Annex for Eurocode – Basis of structural design*

NA to BS EN 1991-2:2003, *UK National Annex to Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges*

3 BS EN 1991-2:2003, Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges

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

A study has been carried out to compare type HA loading that was given in BS 5400-2 (BD 37/01 1) [1]) and Load Model 1 (LM1) in BS EN 1991-2 with the parameters given in the UK National Annex. The LM1 effects are within the acceptable range for spans up to 1 500 m. It is concluded that, assuming type HA loading in BS 5400-2 was suitable for bridges with a span up to 1 500 m, the application of LM1 is appropriate for bridges up to 1 500 m loaded length.

3.2 Adjustment factors α for LM1 [BS EN 1991-2:2003, 4.3.2 (3), Notes 1 and 2]

The values given in the NA to BS EN 1991-2:2003, Table NA.1 have been derived based on calibration against HA serviceability limit state (SLS) loading from BS 5400-2 and verified using reliability analysis (see Atkins' *Background to the UK National Annexes to BS EN 1990 and BS EN 1991-2* [2]). This loading should be used together with the partial factors given in the NA to BS EN 1990:2002+A1.

3.3 Adjustment factor β for LM2 [BS EN 1991-2:2003, 4.3.3 (2)]

The NA to BS EN 1991-2, **NA.2.14** provides for a wheel load of 200 kN compared with the 100 kN load that was given in BS 5400-2:2006, **6.2.5**. However, the wheel contact area has been adjusted as in the

1) The technical content of BS 5400-2:2006 was extracted from BD 37/01.

NA to BS EN 1991-2, **NA.2.15**, which results in similar contact pressures to those previously given in BS 5400-2.

3.4 Wheel contact surface for LM2 [BS EN 1991-2:2003, 4.3.3 (4)]

The wheel contact area given in the NA to BS EN 1991-2:2003, **NA.2.15** results in a characteristic pressure of 1,25 N/mm^2 compared with 1,1 N/mm^2 that was given in BS 5400-2:2006. The corresponding design pressures are 1,69 N/mm^2 and 1,82 N/mm^2. This also makes the wheel contact areas the same for LM1 and LM2.

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

This value of 900 kN has been retained for harmonization although the value was 750 kN in BS 5400-2, **6.10.1**.

NOTE The value of α_{q1}, used to calculate the braking force is given as 1,0 in the note to the NA to BS EN 1991-2:2003, Table NA.1, which differs from the value used for vertical loading. This is because the α_{q1} values needed to calibrate the braking force and vertical loads are different.

3.6 Horizontal forces associated with LM3 [BS EN 1991-2:2003, 4.4.1 (3)]

3.6.1 Longitudinal braking force [NA to BS EN 1991-2:2003, 2.18.1]

Where the loaded length is less than the length of the *SV* or *SOV* vehicle (see Figures NA.1 to NA.3), only the weight of the axles likely to occupy the loaded length when the vehicle is in its most unfavourable position should be considered in calculating ω. Where a structure is designed for *SV196*, the maximum braking force resulting from either the *SV100* or *SV196* vehicle should be considered. *SV80* and *SV100* vehicles produce approximately the same magnitude of braking force because the increase in the basic axle load ω is counteracted by the decrease in the deceleration factor δ.

3.6.2 Centrifugal force [NA to BS EN 1991-2:2003, 2.18.2]

Where the loaded length is less than the length of the *SV* or *SOV* vehicle, only the weight of the axles likely to occupy the loaded length when the vehicle is in its most unfavourable position should be considered in calculating *W*. All three *SV* vehicle models produce the same level of centrifugal force for loaded lengths over 14 m, and it is therefore sufficient to consider only one vehicle in this case. However, for loaded lengths under 14 m, differences might arise due to the number of axles present and the axle weights. The centrifugal force should therefore be checked for different vehicles.

The horizontal forces should be combined with the vertical loads for *SV* or *SOV* vehicles as given in Table NA.3 for Group 6.

NOTE The basis for this can be found in the Background to the UK National Annexes to BS EN 1990 and BS EN 1991-2 [2].

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

The horizontal force transmitted by expansion joints given in BS EN 1991-2:2003 of 180 kN/axle should be used as there was no equivalent value given in BS 5400-2. It is not dissimilar to the value of 180 kN/m run of the joint given in BD 33/94 [3] given that the UDL from each horizontal wheel force of 90 kN applied over a 400 mm width is 225 kN/m.

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

The transverse force due to skew braking or skidding, Q_{trk}, should be considered to act simultaneously with Q_{lk}.

BS EN 1991-2:2003 recommends a value of 25% of Q_{lk} which is approximately 90 kN while BS 5400-2:2006, **6.11.1** gave a constant force for skidding of 300 kN. The latter is considered to be excessive.

3.9 Fatigue load models [BS EN 1991-2:2003, 4.6]

Background to the fatigue load models can be found in the *Derivation of the UK National Annex to Clause 4.6: Fatigue Load Models* [4].

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

No modification is proposed as the recommended values are reasonably close to those that were given in BS 5400-10 [2)].

3.11 Fatigue LM4 [BS EN 1991-2:2003, 4.6.1 (2) Note 2(e) and 4.6.5 (1) Note 2]

Fatigue LM4 consists of sets of standard lorries that together produce effects equivalent to those of real traffic. The NA to BS EN 1991-2:2003, Table NA.5 defines the properties of standard lorries for use in fatigue design on all routes.

The characterization of road traffic for fatigue design purposes depends on the traffic lane configuration, traffic flow rates, proportions of heavy goods vehicles, types of heavy goods vehicles and vehicle usage. Traffic flow rates are route-dependent and the NA to BS EN 1991-2:2003, Table NA.4 should be used to determine the traffic flows. However, road provision is made approximately in relation to demand, and it is recommended that the highway traffic flow rates that are presented in the NA to BS EN 1991-2:2003, Table NA.4 be adopted for design as stated in the National Annex.

[2)] Now withdrawn and superseded by BS EN 1993-1-3.

3.12 Fatigue LM5 (based on recorded traffic data) [BS EN 1991-2:2003, 4.6.6 (1)]

Fatigue LM5 consists of the direct application of recorded traffic flows at specific sites. The traffic flow data for the development of such a model can be supplemented, if appropriate, by statistical extrapolation.

The following guidelines should be used in the development of a site-specific fatigue load model.

- Limited data indicates that the nature of use and, therefore, the weight distributions of each type of heavy goods vehicle appear to depend more on the country (and thus the legislative and vehicle control frameworks) than to sites within individual countries.
- Certain sites are particularly subject to asymmetric flows and vehicle populations might contain unusually large fractions of either fully laden or empty vehicles. Such sites might include access routes to mines or quarries, to large industrial locations and to port facilities.
- It is most important to accurately model the numbers of the heavier goods vehicle types. Estimates of the "total goods vehicle flow" can therefore be very misleading, owing to the variable means of monitoring and classifying twin axle lorries. Care should be taken to accurately estimate the number of lorries with three or more axles, and to correctly match the numbers of such vehicles to those in general traffic models where these are used.

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

3.13.1 Global effects, 4.7.3.3 (1)

The magnitude of forces transferred to a structure during vehicle collision against a vehicle restraint system depends on a complex interaction between the speed of impact, mass and stiffness of the vehicle, the strength and stiffness of the restraint system, the stiffness of the connection between the vehicle restraint system and the structure and the stiffness of the structure. For design purposes simple rules are required that take account of the key parameters.

In BS 5400-2 the forces were based on the containment level and mass of the structure. In BS EN 1991-2 the forces are based on the stiffness of the connection. However, it was felt that designers could have difficulty in determining whether the stiffness of a connection is weak or strong. The approach recommended in the National Annex takes into account only the containment level and the stiffness of the parapet, which can be more readily determined by the designers. The magnitude of forces given in the NA to BS EN 1991-2:2003, Table NA.6 is broadly in line with those that were specified in BS 5400-2:2006, **6.7**.

The transverse forces in the NA to BS EN 1991-2:2003, Table NA.6 correspond to the values given in BS EN 1991-2, Table 4.9 (n) while the longitudinal and vertical forces were based on BS 5400-2:2006, **6.7** with partial factors set to unity. It is possible that BS EN 1991-2 does not take into account the longitudinal and vertical forces as they were assumed not to have a significant influence on the design.

Note that BS 5400-2, **6.7.2** did not take into account global forces for all parapets other than the very high containment parapets. Therefore an amendment to the NA to BS EN 1991-2:2003, **NA.2.30** has been prepared. The proposed amendment is that the last paragraph of the NA to BS EN 1991-2:2003, **NA.2.30.1** should be amended as follows:

For classes A and B the transverse forces in Table NA.6 should be applied 100 mm below the top of the vehicle restraint system or 1,0 m above the level of carriageway or footway, whichever is lower, and on a line 0,5 m long. For classes C and D, 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 construction.

It is recommended that the appropriate class of forces that should be used is agreed on a project specific basis.

The position of the application of the vertical force has been based on BS 5400-2:2006, **6.7.2.1**. The use of the vertical force, which models a wheel climbing the parapet, makes sense only at the top of the parapet.

The co-existing loading due to normal traffic given in BS EN 1991-2, **4.7.3.3** (1), Note 3, ($0{,}75\alpha_{Q1}Q_{1k}$) is considerably lower than the full type HA loading + the accidental vehicle load that was specified in BS 5400-2:2006, **6.7.2.2**. The latter is obviously more conservative for use in an accidental situation and, therefore, the requirement in BS EN 1991-2 might appear to be less conservative. For this reason a reduction factor of 0,75 on the lane 1 loading for LM1 ($0{,}75\alpha_{Q1}Q_{1k}$), which is the frequent value, in addition to the accidental vehicle loading, has been adopted in the NA to BS EN 1991-2:2003, **NA.2.30.2**. This is considered to give an adequate representation of the design situation.

3.13.2 Local effects, 4.7.3.3 (2)

The approach given in the National Annex in essence follows BS EN 1991-2, supplemented with additional guidance that was contained in BS 5400-2. The underlying principle of the two standards is the same.

The magnitude of the collision forces calculated using the National Annex will not be the same as those previously found in BS 5400-2. The National Annex uses characteristic resistance multiplied by 1,25 [BS EN 1991-2:2003, **4.7.3.3** (2)], while BS 5400-2 used design resistance (which was lower than the characteristic) multiplied by a partial factor of 1,4 or 1,5 (BS 5400-2:2006, **6.7.1.4**).

1,25 times the characteristic resistance gives the approximate mean strength for a concrete parapet and is slightly higher than the mean for a metal parapet post. This is likely to be adequate since the supporting member will be designed with a partial factor on its resistance.

No other loading is considered in BS EN 1991-2 to act simultaneously with the horizontal collision forces for local effects, while BS 5400-2 considered the coexistence of the vertical loads attributable to the accidental vehicle loading. The latter approach has been adopted in the National Annex as the two sets of loads are bound to coexist during vehicle collision with a parapet (see the NA to BS EN 1991-2, **NA.2.30.2**).

The accidental vehicle loading given in BS EN 1991-2 is considerably lighter than that given in BD 21/01 [5], the assessment standard for

UK highway structures, and the former design standard BS 5400-2. Designers should consider designing bridges for higher accidental wheel per vehicle loads for individual projects where the likelihood of such loading is considered significant. The following paragraph should be considered in addition to the NA to BS EN 1991-2:2003, **NA.2.30.2** where a higher accidental wheel per vehicle load is considered to be appropriate.

Members supporting central reserves, outer verges and footways, which are not protected from vehicular traffic by an effective barrier, should be assessed for an enhanced accidental vehicle loading in addition to the requirements in BS EN 1991-2. For the purposes of considering this accidental vehicle loading, a new effective carriageway should be defined, extending between bridge parapets or between other effective barriers. LM1, comprising a single tandem system and 3 m width of UDL, should be applied in whatever lateral or longitudinal position produces the most adverse effect on the element considered. Where the application of any wheel or wheels has a relieving effect, it or they should be ignored. Smaller widths of UDL should also be considered where this is more adverse than the full 3 m width. Lane factors for Lane 1 should be applied to the loading. This additional load case should be considered in the accidental combination of actions and should not be considered in conjunction with other variable actions.

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

The choice of the parapet loading class depends on the particular situation of the structure and is normally project specific.

In situations where crowd loading is expected, e.g. access to a sports stadium, it would be normal to specify a parapet with a load class of at least Class H (2,8 kN/m) in accordance with Table 1.

Table 1 **Loading classes**

Class	**Characteristic value of horizontal uniformly distributed load** kN/m
A	0,4
B	0,8
C	1,0
D	1,2
E	1,6
F	2,0
G	2,4
H	2,8
J	3,0

3.15 Model for vertical loads on abutments and wing walls adjacent to bridges [BS EN 1991-2:2003, 4.9.1]

3.15.1 Loading from normal traffic [NA to BS EN 1991-2:2003, NA.2.34.2]

The vehicle configuration given in the NA to BS EN 1991-2:2003, Figure NA.6 corresponds to the 32 tonne rigid vehicle among the set of vehicles given in BD 21/01, Annex D [5].

This vehicle is the most compact and has the highest load density (weight/length is 3,8 tonnes/m) among the set of vehicles. This vehicle produces the most severe bending and shearing effects for loaded lengths up to 15 m.

The dynamic amplification factor and the overload factors have been derived by calibrating the average ultimate limit state (ULS) load effect from the model vehicle against the HA loading effect, found in BS 5400-2, for bending and shearing in a simply supported beam for loaded lengths of between 10 m and 15 m.

3.15.2 Loading from special vehicles [NA to BS EN 1991-2:2003, NA.2.34.3]

The 0,75 factor on the loading from associated normal traffic represents its frequency value. The SV-TT vehicle (military tank transporter) used for the assessment of structures in accordance with BD 86/07 [6], is not currently considered for design loading. It has been assumed that the SV-TT with its 4-wheel 250 kN rear axles is less critical as military vehicles do not overload to the same extent as commercial abnormal vehicles and also because of their 3,5 m width, they displace associated loading from the adjacent lane.

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

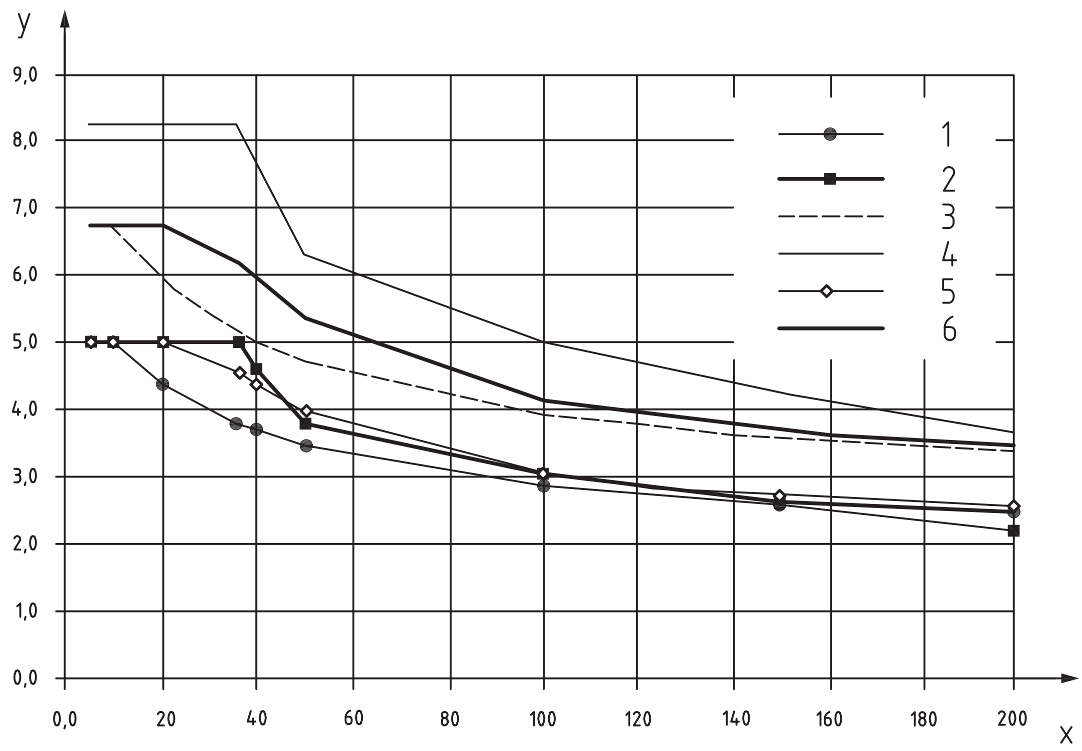
The load model for inspection gangways within road bridges has been taken as in BS EN 1991-2 since an equivalent model was not given in BS 5400-2.

3.17 Uniformly distributed load for road and rail bridges supporting footways or cycle tracks and for footbridges [NA to BS EN 1991-2:2003, NA.2.36]

The recommended values for footbridge loading (for crowd loading and reduced pedestrian loading) are contained in BS EN 1991-2:2003, **5.3.2.1** (2). It is anticipated that, in an amendment to BS EN 1991-2, the uniformly distributed load q_{fk} given in the NA to BS EN 1991-2:2003, **NA.2.36** is to be adopted for BS EN 1991-2:2003, **5.3.2.1** (2) (footbridges) as well as **5.3.2.1** (1) (footways on road bridges). The same loading (**NA.2.36**) is also to be used for footways on rail bridges.

The basis for **NA.2.36** can be found in the *Background to the UK National Annexes to BS EN 1990 and BS EN 1991-2* [2]. The characteristic value given in the NA to BS EN 1991-2:2003, **NA.2.36** is calibrated against the nominal loading that was given in BS 5400-2:2006, **6.5.1**. However, this value is to be used with an SLS partial factor of 1,0 and a ULS partial factor of 1,35 as given in the NA to BS EN 1990:2002, **NA.2.3.9** and **NA.2.3.7** respectively. The pedestrian loadings specified (but excluding crowd loading) by the different standards are compared in Figure 1.

Figure 1 **Comparison of pedestrian load intensity (excluding crowd loading) variation with loaded length for road (and rail) bridges supporting footways or cycle tracks and for footbridges**



x - Loaded length (m)

y - Load intensity (kN/m^2)

Key

1 BS EN 1991-2 nominal A)

2 BS 5400-2:2006 nominal

3 BS EN 1991-2 ULS B)

4 BS 5400-2:2006 ULS

5 NA to BS EN 1991-2 nominal C)

6 NA to BS EN 1991-2 ULS

A) For footbridges only [see BS EN 1991-2:2003, **5.3.2.1** (2)]. For road or railway bridges supporting footways or cycle tracks [see BS EN 1991-2:2003, **5.3.2.1** (1)], the recommended nominal pedestrian load intensity is a constant of 5 kN/m^2.

B) For footbridges only [see BS EN 1991-2:2003, **5.3.2.1** (2)]. For road or railway bridges supporting footways or cycle tracks [see BS EN 1991-2:2003, **5.3.2.1** (1)], the recommended pedestrian load intensity at ULS is a constant of 6,75 kN/m^2.

C) For road or railway bridges supporting footways or cycle tracks (see NA to BS EN 1991-2:2003, **NA.2.36**).

The model in the NA to BS EN 1991-2 provides a good fit to the nominal loading that was given in BS 5400-2:2006, **6.5.1**. A pedestrian load intensity of 5,0 kN/m^2 is considered to represent a very dense crowd. Values greater than 7,0 kN/m^2 would be inconceivable, except where there is the possibility that people are stationary on the bridge, although the dynamic increment of loading can reduce with the extent of crowding.

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

The BS EN 1991-2:2003 value has been adopted as there was no equivalent load model in BS 5400-2.

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

The BS EN 1991-2:2003 value has been adopted since an equivalent load model was not available in BS 5400-2.

3.20 Dynamic models for pedestrian loads on footbridges [BS EN 1991-2:2003, 5.7 (3)]

The values of reduction factor given by the NA to BS EN 1991-2:2003, Figure NA.9 are considered to be erroneous in some cases. A replacement figure has been prepared to address this (see Figure 2).

It is recommended that values of the reduction factor should be agreed on a project specific basis.

The background to the vibration serviceability requirements for pedestrian footbridges is given in Annex A and further information can be found in *Design Methodology for Pedestrian Induced Footbridge Vibration* [7].

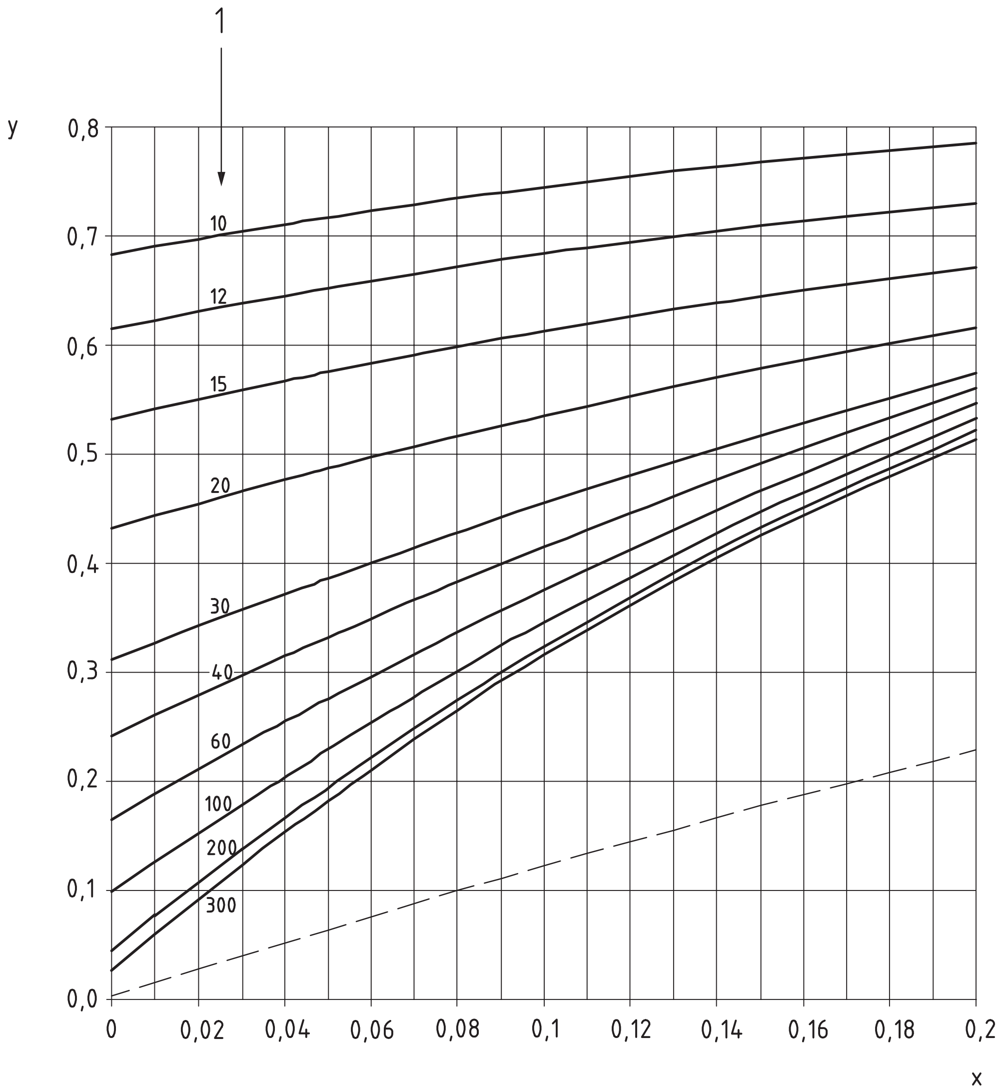
3.21 LM3: Models of special vehicles [BS EN 1991-2:2003, Annex A]

The SV vehicle models have been developed based on the previous work carried out for the development of BD 86/07 [6]. The *SV196* vehicle is slightly more onerous than the assessment SV-Train vehicle but this avoids the need for considering different vehicle types for the design of a structure. Further information on the development of SV models is given in the Atkins' *Background to the UK National Annexes to BS EN 1990 and BS EN 1991-2* [2].

In common with the SV vehicle models in BD 86/07 [6], the *SV196* model has the following limitations.

- The case of two or more STGO vehicles occurring simultaneously within a lane over a bridge is not accounted for since it is not permitted under the STGO Regulations.
- The simultaneous occurrence of two or more abnormal vehicles in adjacent lanes over a bridge is not accounted for since it is not permitted under the STGO Regulations.
- The dynamic amplification factors have been determined based on work undertaken for the development of BD 86/07 [6].

Figure 2 **Reduction factor, λ, to allow for the unsynchronized combination of pedestrian actions within groups and crowds**



x - Structural damping – logarithmic decrement, δ

y - Reduction factor on effective number of pedestrians, γ

Key

1	Effective span, Seff (m)
———	Pedestrian groups
– – – – –	Crowd loading

Annex A (informative) Vibration serviceability recommendations for foot and cycle track bridges

A.1 General

For superstructures, where the fundamental natural frequencies of vibration exceed 8 Hz for the unloaded bridge in a vertical direction and 1,5 Hz for the loaded bridge in a horizontal direction, the necessary vibration serviceability can be deemed to be satisfied.

In general, a linear elastic model is appropriate for calculation of the structure's response to dynamic actions. Nevertheless, attention is drawn to the fact that the constructed footbridge might have different natural frequencies from those calculated (e.g. due to the interaction between structural and non-structural parts) and therefore might respond differently from predictions.

Typical values for the logarithmic decrement of decay of vibration (δ) (structural damping) to be used in design are provided in Table A.1. Structural damping is extremely difficult to predict and bridge fittings, such as parapets, can significantly alter these values. Therefore, it is suggested that the intrinsic damping be measured before providing additional damping to a constructed bridge. The value of structural damping might also depend on the mode and load case under consideration and this needs to be taken into account during design.

Table A.1 **Typical values of logarithmic decrement of decay of vibration (δ) between successive peaks**

Material of construction	Logarithmic decrement of decay of vibration (δ)
Steel	0,03
Steel and concrete composite	0,04
Concrete	0,05
Timber	0,06 to 0,12
Aluminium alloy	0,02
Glass or fibre reinforced plastic	0,04 to 0,08

At the time of publication, research suggests that in crowded conditions the mass of pedestrians might also act to add vertical damping to the bridge and so further reduce the response to dynamic actions from pedestrians. If this effect is proven to be a reliable source of energy dissipation, the extra damping provided may be added to the estimate of structural damping and, in all other respects, the response to dynamic actions from pedestrians calculation will be unaltered. However, under no circumstances should such additional damping be used in the avoidance of unstable lateral responses.

The pedestrian actions described in the NA to BS EN 1991-2, **NA 2.44** correspond to an average pedestrian weight of 700 N. In general, the dynamic action imposed by the excitation of a bridge structure as pedestrians move over it, is proportional to the average weight. For bridges that are to be designed for locations where the national average weight is known to be different to that of the UK, consideration might be given to adjusting the applied actions accordingly.

A.2 Background to “Dynamic actions representing the passage of single pedestrians and pedestrian groups” [NA to BS EN 1991-2:2003, NA.2.44.4]

The dynamic actions represent a group of *N* pedestrians where the following applies.

- Pedestrians in a group make a single crossing of the bridge together.
- One pedestrian in the group is assumed to walk with a pace frequency that is exactly matched to the frequency of the mode being investigated (the reduced likelihood of this occurring in practice is dealt with subsequently by the factor $k(f_v)$ obtained from the NA to BS EN 1991-2:2003, Figure NA.8).
- All other pedestrians in the group (N-1) are assumed to walk with phase and pace rates that are randomly chosen from the pedestrian population model.
- In order to allow for the increased likelihood that pedestrians in small groups might walk with frequencies that are similar to each other, the standard deviation of the distribution of frequencies used for the group pedestrian model has been chosen to be 0,1 [as opposed to the value of 0,3 that was used in the development of the crowd model and the factor $k(f_v)$].

In this way the group model aims to provide an assessment of the effect of groups of pedestrians who are walking more purposefully than those represented by the crowd model.

A.3 Background to “Steady state modelling of pedestrians in crowded conditions” [NA to BS EN 1991-2:2003, NA.2.44.5]

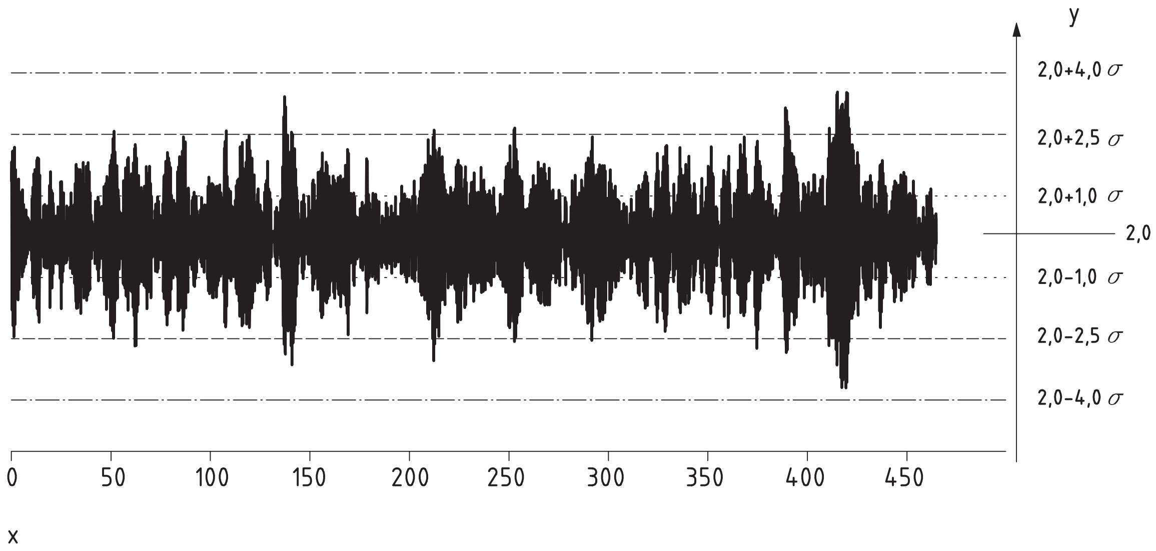
In order to describe the range of possible bridge responses to dynamic actions from pedestrians, which might be caused by crowd loading, it is normal to describe the responses in a probabilistic manner. A useful measure of the amplitude of the response is the root mean square (RMS) of the population of possible responses. As this amplitude is frequently exceeded by a large amount, the RMS is not suitable for comparison with comfort criteria.

NOTE In the extreme there is a real, but very small risk that everyone within a crowd might be fully synchronized for the duration of the loading.

Mathematical techniques are readily available to obtain maximum design values for given return periods. For a bridge response frequency of approximately 2 Hz, it can be shown that 4,0 times the standard deviation (σ) needs to be used to provide a maximum design value with an average return period of 15 minutes. At face value this appears to be a suitable maximum design value. However, if real response signals are examined, it is evident that the use of 4,0 times the standard deviation would be unnecessarily severe.

Figure A.1 illustrates a synthetic time history produced from a random bridge response population model centred on a bridge response frequency of 2 Hz. Horizontal lines mark the boundaries of $\pm$4,0, $\pm$2,5 and $\pm$1,0 times standard deviations for this typical 7½ minute time history.

Figure A.1 **Synthetic time history for a random bridge/pedestrian model**



x - Time (seconds)

y - Bridge response frequency (Hz)

Key

σ Standard deviation

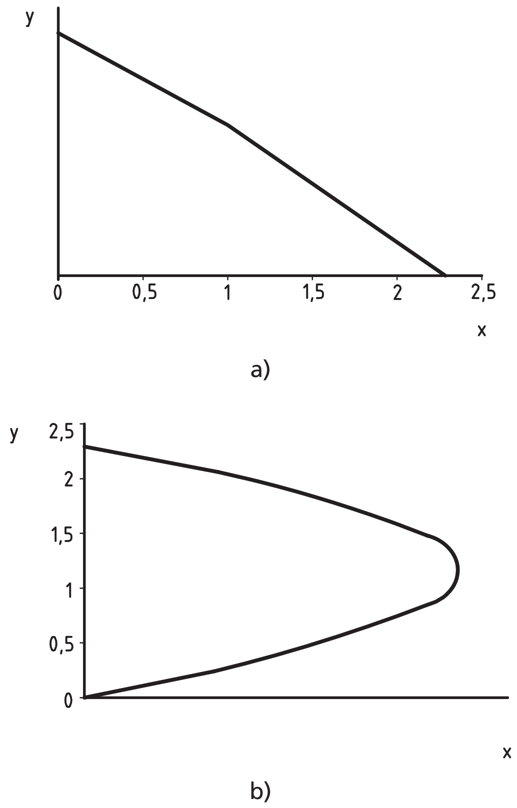
It has been decided that 2,5 times standard deviation is an appropriate maximum design value for use for bridge responses to crowd loading, and the loading provided by the NA to BS EN 1991-2, **NA.2.44.4** incorporates this assumption. This permits short durations of response that are above the target comfort criteria but the average level of comfort is significantly better than the RMS value.

Other design guides (such as the Sétra Technical Guide for the *Assessment of vibrational behaviour of footbridges under pedestrian loading* [8]) use maximum design accelerations that are equivalent to the 4,0 standard deviations illustrated in Figure A.1. In certain circumstances, subject to agreement on a project specific basis, a more stringent limit to the maximum bridge response may be applied by increasing the design maximum loading from 2,5 to 4,0 times standard deviations.

In very crowded conditions, the mean velocity of pedestrians tends to reduce as the density increases (see Figure A.2) and the population statistics change. In the calculation of vertical bridge responses to the NA to BS EN 1991-2, **NA.2.44.4**, the maximum crowd density of 1 persons/m^2 is used to take account of this effect.

There is some evidence to suggest that additional damping might be available in very crowded conditions. In addition, there is rather more evidence to suggest that simple probabilistic models of crowd behaviour, such as that used in the NA to BS EN 1991-2, **NA.2.44.4**, tend to produce calculated bridge responses that are somewhat larger than field measurements of comparable cases. However, at the time of publication, such measured responses cannot be relied upon for design.

Figure A.2 **Reduction of pedestrian speed and flow rates with density**



a) x - Density (people/m^2)

y - Speed

b) x - Flow rate

y - Density (people/m^2)

A.4 A simplified method for deriving the maximum vertical acceleration due to single pedestrians and pedestrian groups

This method is valid only for a single span or two or three-span continuous, symmetric superstructures of constant cross-section and supported on bearings that can be idealized as simple supports.

The maximum vertical acceleration α (m/s^2) can be taken as:

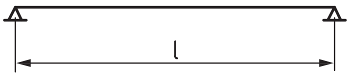
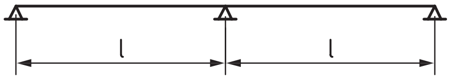
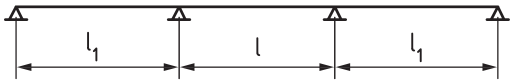
$$\alpha = \left(\frac{F'}{M_i} \gamma_{i\max}^2 \right) K \Psi$$

where:

- F' is the amplitude of the moving dynamic load (N) given in the NA to BS EN 1991-2, **NA.2.44.4** (1). That is:

  $F' = F_0 k\left(f_v\right)\sqrt{1+\gamma\left(N-1\right)}$;
- M_i is the generalized mass of the mode of interest i (kg);
- $\gamma_{i\max}$ is the maximum vertical component of mode shape I;
- K is the configuration factor (see Figure A.3);
- Ψ is the dynamic response factor (see Figure A.4).

Figure A.3 **Configuration factor *K***

1		*K*
	—	1,0
	—	0,7
	Ratio l_1/l	
	1,0	0,6
	0,8	0,8
	0,6 or less	0,9

Key

1 Bridge configuration

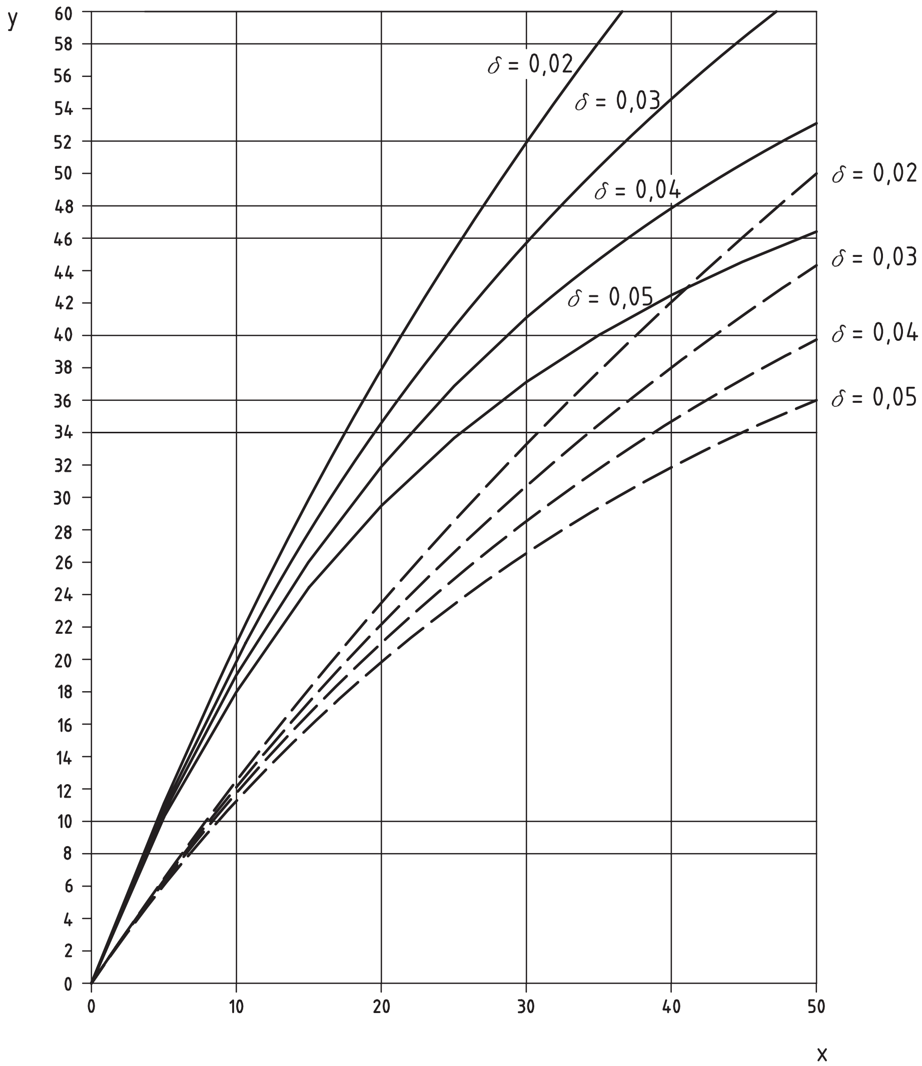
l length of main span (m)

l_1 length of side spans (m)

NOTE For three-span continuous bridges, intermediate values of K may be obtained by linear interpolation.

In all other situations, the maximum vertical acceleration for single pedestrians and pedestrian groups is to be determined from a time history calculation of the bridge response, which explicitly models fluctuating loads passing across the span using the parameters described in the NA to BS EN 1991-2, **NA.2.44.4**.

Figure A.4 **Dynamic response factor Ψ**



x - Main span length (m)

y - Dynamic response factor Ψ

Key

—— Walking

— — Jogging

NOTE 1 Main span length, l, is indicated in Figure A.3.

NOTE 2 Values of δ, the logarithmic decrement of decay of vibration for different types of construction are given in Table A.1.

A.5 A method for deriving the maximum vertical acceleration in crowded conditions

This method is generally applicable.

The maximum vertical acceleration α (m/s^2) can be taken as:

$$\alpha = \left(\frac{\int_0^S w'b\left|\gamma_i\left(x\right)\right|dx}{M_i} \gamma_{i\max} \right) \frac{1}{2\xi}$$

where:

- w' is the amplitude of the vertical dynamic load (N/m^2) given in the NA to BS EN 1991-2, **NA.2.44.3**(b)(1). That is:

$$w' = 1.8\left(\frac{F_0}{A}\right)k\left(f_v\right)\sqrt{\gamma N/\lambda};$$

- b is the width of the walking surface of the bridge (m);
- M_i is the generalized mass of the mode of interest *i* (kg);
- $\gamma_i\left(x\right)$ is the vertical component of mode shape *i* at position *x* (m) along the span *S* (m);

  NOTE In this equation the absolute, positive value of the mode amplitude is used in the integration.

- $\gamma_{i\max}$ is the maximum vertical component of mode shape *I*;
- ξ is the structural damping when expressed as a damping ratio: $\xi = \frac{\delta}{2\pi}$;
- δ is the logarithmic decrement of decay of vibration between successive peaks.

For uniform width spans, where the mode shape is approximately sinusoidal, the first equation in **A.5** can be further simplified to:

$$\alpha = \left(\frac{0.634w'bs}{M_i} \gamma_{i\max}^2 \right) \frac{1}{2\xi}$$

A.6 Damage from forced vibration

Consideration is to be given to the possibility of permanent damage to a superstructure by a group of pedestrians deliberately causing resonant oscillations of the superstructure. As a general precaution, therefore, the bearings need to be of robust construction with adequate provision to resist upward or lateral movement.

For prestressed concrete construction, resonant oscillation might result in a reversal of up to 10% of the static live load bending moment. Providing that sufficient unstressed reinforcement is available to prevent gross cracking, no further consideration needs to be given to this effect.

A.7 Background to "The avoidance of unstable lateral bridge responses due to crowd loading" [NA to BS EN 1991-2:2003, NA.2.44.7]

The stability condition described in the NA to BS EN 1991-2, **NA.2.44.4** is significantly different from the comfort criteria described in respect of vertical vibrations. While the level of comfort experienced by bridge users is very subjective and varies from person to person and site to site, the lateral stability condition has a measurable and clearly defined limit that is not to be exceeded if large uncontrolled lateral motions are to be avoided.

The process provided in the NA to BS EN 1991-2, **NA.2.44.7** is based on a uniform deck mass. Lower critical densities can arise where the deck mass per unit length is irregular; a conservative approach would be to use the lowest mass per unit length. This method might also be conservative when other parts of the structure, such as arch ribs or suspension cables, add significantly to the modal mass. In these circumstances, specialist advice needs to be sought.

At the time of publication, 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 bridge response only and are to be used with caution.

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