Guidance on Loading Requirements for the Design of Railway Structures

Issue record

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<td>One</td>
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Gives guidance on the loading requirements for the design of railway structures, it also provides guidance on the structures requirements within the latest draft of the Infrastructure Technical Specification for Interoperability (INF TSI).

Superseded documents

The following Railway Group documents are superseded, either in whole or in part as indicated:

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<td>GC/RT5112 issue two, Rail Traffic Loading Requirements for the Design of Railway Structures</td>
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GC/RT5112 issue two ceases to be in force and is withdrawn as of 07 March 2015.

Supply

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Part 1  Introduction

G 1.1  Purpose of this document

G 1.1.1  This document gives guidance on loading requirements for the design of railway structures. It does not constitute a recommended method of meeting any set of mandatory requirements.

G 1.1.2  The coverage of the document is addressed within section G 1.2, which explains the subject areas for which guidance has been provided, for example 'aerodynamics' and 'derailment actions'.

G 1.2  Scope

G 1.2.1  In the context of this document railway structures includes:

a) Under-line bridges.
b) Over-line bridges.
c) Lineside structures.
d) Tunnel inverts.
e) Culverts.
f) Buried structures.
g) Retaining walls and embankments.

G 1.2.2  This document:

a) Lists, by means of a matrix, key documents which set out loading requirements for railway structures, together with other relevant documents.
c) Provides guidance on aerodynamic actions and the requirements of BS EN 1991-2:2003 incorporating corrigenda December 2004 and February 2010, Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges - 6.6 Aerodynamic actions: the guidance may be used with confidence for train speeds up to 225 mph (360 km/h).
d) Provides guidance on accidental actions and the requirements of BS EN 1991-1-7:2006 incorporating corrigendum February 2010, Eurocode 1: Actions on structures - Part 1-7: General actions - Accidental actions - 4.5 Accidental actions caused by derailed rail traffic under or adjacent to structures, the guidance is appropriate for train speeds up to 225 mph (360 km/h).

G 1.2.3  A revised Infrastructure Technical Specification for Interoperability (INF TSI), is planned to replace the current High Speed (HS) INF TSI (Commission Decision 2008/217/EC) and the Conventional Rail (CR) INF TSI (Commission Decision 2011/275/EU), in January 2015. The scope of the revised INF TSI will extend to the whole of the mainline railway system.
Guidance on Loading Requirements for the Design of Railway Structures

G 1.2.4 Guidance is provided in Appendix A of this document on the requirements for structures set out in the revised INF TSI, based on the latest available draft at the time of writing (version 4.5, 23/04/2013). (The requirements for structures set out in the draft of the revised INF TSI are considered unlikely to change significantly before publication of the revised INF TSI.) The guidance is provided as a series of sequentially numbered clauses prefixed ‘G’ immediately below greyed extracts from the draft INF TSI to which it relates. Provision of guidance in this document is an interim measure prior to issue of the revised INF TSI and prior to future amendment of GI/GN7608. This guidance will be transferred to GI/GN7608 when there is an opportunity to do so.

G 1.3 Copyright

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G 1.4 Approval and authorisation of this document

G 1.4.1 The content of this document was approved by Infrastructure Standards Committee on 10 September 2014.

G 1.4.2 This document was authorised by RSSB on 01 October 2014.
Guidance on Loading Requirements for the Design of Railway Structures

Part 2 Source of Design Requirements for Loading of Railway Structures

G 2.1 Document matrix

G 2.1.1 The key documents which set out the loading requirements for the design of railway structures are listed below:

a) INF TSI.


G 2.1.2 Implementation of the European Directive covering procurement procedures for public bodies in the transport sector (Directive 2004/17/EC) will require publicly funded works to be designed to the Structural Eurocodes once the national standards are withdrawn. Directive 2004/17/EC is implemented in the GB through the Utilities Contracts Regulations 2006 and the Utilities Contracts (Scotland) Regulations 2006.

G 2.1.3 Indication of the particular clauses within the listed key documents is given in the form of a matrix (see Table G 1), with design requirements listed along the vertical-axis and relevant documents listed on the horizontal-axis. Cells either indicate the precise clauses in a document that are relevant, or include ‘x’ to indicate that the document should be referred to in its entirety.

G 2.1.4 RGSs which capture vehicle / bridge compatibility requirements related to design of railway structures, and UIC Leaflet 777-2R, 2nd Edition, September 2002, ’Structures built over railway lines - Construction requirements in the track zone’, which provides guidance on design and risk assessment for accidental actions resulting in impact with railway structures, are also included.
### Guidance on Loading Requirements for the Design of Railway Structures

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**Table G 1** Documents for loading requirements for the design of railway structures
G 2.1.5 The design requirements and guidance within the listed documents are primarily intended for the design of bridges, although they are also applicable to all railway structures which are subjected to railway traffic.


G 2.2.1 BS EN 1990:2002 + A1:2005 clause A2.4.4.2.3 requires vertical deformation due to railway traffic actions to be checked with the ‘classified characteristic vertical loading’, for example, including the load classification factor $\alpha$, but makes no mention of the dynamic factor $\Phi$. This is an oversight in the standard and will in due course require an amendment to EN 1990:2002 + A1:2005. For calculation of vertical deformation, the effects from railway traffic should be multiplied by $\alpha$ and $\Phi$.

G 2.3 Dynamically sensitive railway structures

G 2.3.1 For novel and dynamically sensitive railway structures, the European Committee for Standardisation (CEN), has carried out studies in response to a request from the European Railway Agency (ERA), on the compatibility of vehicles and railway bridges across the European Union. This work has identified that supplementary vehicle / bridge compatibility checks are required and that there is a potential problem across all countries for vehicles that operate in the 100 – 125 mph (160 – 200 km/h) range.

G 2.3.2 BS EN 1991-2:2003 Figure 6.9 sets out a flow chart for determining whether a dynamic analysis is required. A dynamic analysis is required where train speeds are greater than 125 mph (200 km/h), and / or the limits of bridge natural frequency ($\eta_0$) fall outside the specified limits, but it does not distinguish between ‘simple’ and ‘complex’ railway structures. Figures NA.2.12 and NA.2.13 in the UK National Annex to BS EN 1991-2:2003 provide alternative flow charts for simple and simple / complex structures respectively, but also require a dynamic analysis for simple structures where train speeds are greater than 125 mph (200 km/h), or for simple / complex structures where train speeds are greater than 110 mph (180 km/h).

G 2.3.3 Loading from vehicles operating outside of the 100 – 125 mph (160 – 200 km/h) speed range, and in some cases between 90 – 100 mph (140 – 160 km/h), has caused increased ‘dynamic effects’, including higher accelerations within the ballast which has the potential to reduce the effectiveness of the ballast in providing support to the track.

G 2.3.4 The higher speeds and increased axle loads attributable to the emerging generation of European multiple units, will result in the occurrence of dynamic effects (acceleration and deformation), which may exceed the design limits in BS EN 1990:2002 + A1:2005 Annex A2. It may therefore be necessary to modify the design requirements for structures which are subject to traffic loading, in particular bridges, by increasing the magnitude of the dynamic factor applied to the static traffic loading and / or examining the suitability of the design limits within BS EN 1990:2002 + A1:2005 Annex A2.

G 2.3.5 Research is being undertaken on behalf of the Vehicle / Structures System Interface Committee (V/S SIC), to investigate this issue for Great Britain (GB) conditions. The project will examine the aspirations of the operators, train manufacturers, and train service specifiers, for current and future GB vehicles and to establish appropriate Reference Load Models (RLMs) to represent them. Additionally, an investigation will be undertaken into the compatibility of the RLMs with GB bridges for speeds in excess of 90 mph (140 km/h), provide guidance on compatibility assessment, and examine the acceptance criteria for ballast acceleration in BS EN 1990:2002 + A1:2005 Annex A2.
Guidance on Loading Requirements for the Design of Railway Structures

G 2.4 Trackside and overhead structures subjected to transient aerodynamic loads

G 2.4.1 RSSB research project T750 ‘Review of Euronorm design requirements for trackside and overhead structures subjected to transient aerodynamic loads’, has concluded that the application of the aerodynamic design requirements for structures in BS EN 1991-2:2003 would result in overestimation of the loads for GB structures, as the design pressures are based on larger continental gauge rolling stock.

G 2.4.2 Alternative design pressures for GB structures have been developed on the basis of train model testing. These alternative design requirements have been included within Part 3 of this document in advance of their publication within a future revision of the UK National Annex for BS EN 1991-2:2003.

G 2.5 Design of railway structures to resist derailment loads

G 2.5.1 Design requirements for railway structures to resist derailment loads are set out in BS EN 1991-1-7:2006 and its UK National Annex. Guidance on undertaking a risk assessment to support a decision to design structures for a reduced loading is set out in Part 4 of this document, for use in circumstances where an alternative design strategy, other than design for the recommended impact values for Class A and B structures, is considered to be appropriate.
Part 3 Guidance on GB Requirements for Design of Structures Subject to Aerodynamic Actions

G 3.1 Introduction

G 3.1.1 In the absence of ‘alternative values’ in the UK National Annex for BS EN 1991-2:2003, RSSB has undertaken research (see research project T750 for further details), which provides the basis for providing the alternative GB specific design requirements to replace the existing clause NA.2.74 of UK National Annex for BS EN 1991-2:2003.

G 3.1.2 Until the UK National Annex for BS EN 1991-2:2003 is updated, the following guidance has been provided to help the industry when considering the design of structures that are subject to aerodynamic actions.

G 3.2 Aerodynamic actions

G 3.2.1 A moving train generates an aerodynamic pressure field, which it imposes on structures alongside and over the track. The pressure field is largest at the front and rear of the train and at the nose-to-nose coupling between separate rakes of multiple units. The resultant pressure field exerts load on trackside structures, which is taken into account in design to ensure that they can withstand the aerodynamic actions (loads) and do not sustain fatigue damage.

G 3.2.2 When calculating the mechanical and fatigue strength of a structure, the aerodynamic actions are assumed to be of an equivalent amplitude at the head and tail and, in the case of nose-to-nose coupled trains, also at the coupler. This amplitude is taken to be the largest of the actions generated during the complete time of passing of the train.

G 3.2.3 In reality, a train passing generates a maximum pressure (aerodynamic action). The maximum load that the structure experiences will depend upon the dynamic sensitivity of the structure and its response to the aerodynamic action. It is recommended that a characteristic pressure value is taken to represent the maximum static action. BS EN 1991-2:2003, clause 6.6.1 (5) recommends a dynamic amplification factor of 2.0. The adequacy of this factor may need to be determined from a structure-specific study taking into account the dynamic response of the structure.

G 3.2.4 Whether a dynamic amplification factor of 2.0 is too high or insufficient, will depend upon whether the structure is able to respond to the pressure wave from a passing train. This is not a judgement that is possible without prior knowledge of the sensitivity of a structure to passing pressure waves or by undertaking a structure-specific dynamic analysis.

G 3.2.5 Guidance on determination of the dynamic amplification for noise barriers is set out in UIC-779-1R, 2nd Edition ‘Effect of the slipstream of passing trains on structures adjacent to the track’.

G 3.2.6 These aerodynamic actions are applied perpendicular to the surface of the structure in each case (see Figures G 1 to G 3 of this document). They are termed characteristic pressure values and it should be noted that in most cases the given characteristic pressure values (or equivalent pressures in non-dimensionalised form), represent the maximum area-averaged pressures on a structure. The design characteristic pressure distributions in Figures G 1 to G 3 of this document are relevant for application to the surfaces indicated for each structure type. Pressure distributions have not been derived for vertical surfaces above the track such as bridge parapet fences.

G 3.2.7 Guidance on the susceptibility of bridge structures to aerodynamic excitation, including the provision of approximate formulae for estimation of the fundamental frequencies of bending and torsion for bridge structures is set out in the Highways Agency standard BD 49/01 ‘Design rules for aerodynamic effects on bridges’. However, its application for consideration of railway bridge structures subject to the aerodynamic effects from passing trains, is expected to be restricted to ‘limited amplitude response’, due to the smaller magnitude and frequency of aerodynamic actions compared to ambient wind.
Guidance on Loading Requirements for the Design of Railway Structures

G 3.2.8 Guidance on the assessment of fatigue sensitive details for noise barriers is set out in UIC 779-1R, 2nd Edition 'Effect of the slipstream of passing trains on structures adjacent to the track’. No specific guidance is available for fatigue-sensitive details associated with other structure types.

G 3.2.9 For structures that are dynamically sensitive, fatigue calculations require an assessment of the additional number of pressure cycles generated by the dynamic response of the structure arising from the train-induced pressure pulse. The magnitude of the pressure pulse increases with reduction of the length of structure over which the pressure pulse is assumed to act. The design pressures for local fatigue sensitive details may be critical in some cases.

G 3.2.10 The resulting equivalent pressures that are calculated from the characteristic pressure values are mainly dependent on:
   a) The square of the speed of the train.
   b) The aerodynamic shape of the front of the train.
   c) The shape and type of the structure.
   d) The position of the structure, in particular the clearance between the vehicle and the structure.

G 3.2.11 Train aerodynamic nose shapes are classified here as bluff, intermediate or streamlined corresponding to freight locomotives, standard GB multiple units and streamlined high speed trains.

G 3.3 Relationship with BS EN 1991-2:2003

G 3.3.1 In BS EN 1991-2:2003, section 6.6 ‘Aerodynamic actions from passing trains’, methods are provided for calculating equivalent pressure loads on a variety of trackside structures. These can be used, but will generally over-estimate the equivalent loads, as the data on which the code is based used continental gauge trains, which lead to reduced clearances between the vehicles and the structures. However, the method for simple horizontal structures adjacent to track in BS EN 1991-2:2003, clause 6.6.4 is not recommended, as GB platforms are higher than European platforms and under-estimates of the pressure loads may result.

G 3.3.2 The data on which the following GB specific guidance for alternative design pressures is based forms a consistent and extensive database and was derived from moving model tests using GB gauge trains. It is therefore considered to be more appropriate for calculating equivalent pressures on relevant GB trackside structures.

G 3.4 Flat vertical structures parallel to the tracks

G 3.4.1 The following structures belong to this category:
   a) Trackside acoustic and wind protection barriers.
   b) Walls and fences.
   c) Facades of buildings near the track.
   d) Trackside and platform hoardings.

G 3.4.2 The true pressure field around the train is considered to be represented by the equivalent distributed pressures $+p_1k$ and $-p_1k$, each of which is taken to be 5 m long and proportional to the speed of the train as shown in Figure G 1 of this document. The calculated pressure applies from the foot of the structure on the track formation, up to a maximum height of 5 m above rail level. The pressure distributions apply at the train nose and tail, as well as to intermediate nose-to-nose couplings if present.
Figure G 1  Pressure distributions on a vertical structure next to the track

G 3.4.3 The equivalent pressures are determined from equation (E3.1):

\[ p_{1k} = 0.5 \rho v^2 k_1 C_{p1}(Y) \]  \( \text{(E3.1)} \)

Where \( C_{p1} \) is the characteristic pressure coefficient value depending on the distance to the structure from track centre \( Y \), and \( k_1 \) is the shape parameter of the train. (See Figure G 4 of this document for details of the co-ordinate system used).

Where \( k_1 = 1.0 \) for bluff trains for all trackside and platform mounted structures.

\( k_1 = 0.613 \) for intermediate trains and \( 0.432 \) for streamlined trains for trackside structures.

\( k_1 = 0.85 \) for intermediate trains and \( 0.63 \) for streamlined trains for platform mounted structures.

\( C_{p1} \) is obtained from equation (E3.2) for trackside structures and equation (E3.3) for platform mounted structures.

\[ C_{p1}(Y) = \pm \frac{8.0}{(Y+2.10)^2} \text{ for } Y \geq 1.45 \text{ m} \]  \( \text{(E3.2)} \)

\[ C_{p1}(Y) = \pm \left( \frac{8.0}{(Y+2.32)^2} + 0.1 \right) \text{ for } Y \geq 1.95 \text{ m} \]  \( \text{(E3.3)} \)

G 3.4.4 For small structural elements up to 1.0 m high or up to 2.5 m long, the pressure, \( p_{1k} \), should be increased by a factor of 1.3.

G 3.5 Simple horizontal structures above the track

G 3.5.1 Structures over the railway, for example bridge decks and access structures belong to this category.

G 3.5.2 The true pressure field around the train is considered to be represented by the equivalent distributed pressures \( +p_{2k} \) and \( -p_{2k} \) each of which is 5 m long and proportional to the speed of the train as shown in Figure G 2 of this document. These pressures are a maximum at the centre of the vehicle over the track and reduce with lateral distance either side.
Figure G 2  Pressure loads on a horizontal structure above the track

G 3.5.3 The equivalent pressure loads are determined from equation (E3.4):

\[ p_{2k} = 0.5 \rho v^2 k_2 C_{p_2}(h, W, y) \]  

(E3.4)

Where \( C_{p_2} \) is the characteristic pressure coefficient value depending on the height, \( h \), of the structure above rail level, the along track width, \( W \), of the structure and the lateral distance, \( y \), from the track centreline, and \( k_2 \) is the shape parameter of the train.

Where \( k_2 = 1.0 \) for bluff trains, 0.432 for intermediate and streamlined trains.

G 3.5.4 The characteristic pressure coefficient values for 10 m wide structures of height, \( h \), and at the track centreline (\( y=0 \)) are given by the following:

\[ C_{p_2}(h, 10, 0) = \pm \left( \frac{5.5}{(h-1.9)^2} + 0.1 \right) \]  

(E3.5)

G 3.5.5 Using the calculated value for \( C_{p_2}(h,10,0) \), the variation with structure width, \( W \), for bluff, intermediate, and streamlined trains is respectively given by:

\[ C_{p_2,\text{bluff}}(h, W, 0) = (0.025W + 0.75)C_{p_2}(h, 10, 0) \]  

(E3.6a)

\[ C_{p_2,\text{int,str}}(h, W, 0) = 0.51C_{p_2,\text{bluff}}(h, W, 0) \], for \( 1.5 \text{ m} \leq W \leq 3.0 \text{ m} \)  

(E3.6b)

\[ C_{p_2,\text{int,str}}(h, W, 0) = 0.51C_{p_2,\text{bluff}}(h, 3.0) \], for \( 3.0 \text{ m} < W < 20.0 \text{ m} \)  

(E3.6c)

G 3.5.6 The across track variation, \( y \), of the pressure value \( C_{p_2}(h, W, y) \) can be obtained from the calculated \( C_{p_2}(h, W, 0) \) values for each train type. The pressure variation takes the form:

\[ C_{p_2}(h, W, y) = C_{p_2}(h, W, 0)(1 - 0.03y^2) \], for \( y < 5.8 \text{ m} \)  

(E3.7a)

\[ C_{p_2}(h, W, y) = 0, \text{ for } y \geq 5.8 \text{ m} \]  

(E3.7b)

G 3.5.7 In the case of passing trains, the pressures from each train are superimposed. However, there is no need to consider more than two tracks.

G 3.5.8 The pressures acting on the edge strips of a wide structure which cross the track may be multiplied by a factor of 0.75 over a width of 1.50 m.

G 3.5.9 For structures, for which \( W \geq 20 \text{ m} \), closed structures above the track can be considered as tunnels. In such cases, train-induced pressure waves become important and need to be considered in the determination of pressures on the structure.
G 3.6 Platform canopies and horizontal surfaces adjacent to the track

G 3.6.1 The following structures belong to this category:

a) Platform canopies with a minimum height of 4 m above rail level and a back wall at a minimum distance of 3.45 m from the track centreline.

b) Platform canopies with a minimum height of 4 m above rail level with no back wall and without the blockage caused by a stationary train located on the track adjacent to the platform edge furthest away from the passing train.

G 3.6.2 The true pressure field around the train is considered to be represented by the equivalent distributed pressures \( +p_{3k} \) and \( -p_{3k} \) each of which is 5 m long and proportional to the speed of the train as shown in Figure G 3 of this document. These pressures are a maximum at the edge of the canopy closest to the running track and reduce with increasing lateral distance.

G 3.6.3 The equivalent pressures for canopies of height \( h \) above track with back walls located at \( Y \) from the track centreline are determined from equation (E3.8):

\[
p_{3k} = 0.5 \rho v^2 k_3 C_{p3}(h,Y)
\]  

(E3.8)

G 3.6.4 The characteristic pressure coefficient values are given by equation (E3.9):

\[
C_{p3}(h,Y) = \pm \left( \frac{6.8}{(h-0.1)} \right) \left( 1 - 0.13(Y - 3.45)^2 \right)
\]  

(E3.9)

Where \( k_3 = 1.0 \) for bluff trains, 0.53 for intermediate trains and 0.43 for streamlined trains.

For back wall distances greater than 5 m from track centreline, \( C_{p3} \) should be evaluated with \( Y = 5 \) m.

G 3.6.5 The characteristic pressure coefficient values for canopies with no back walls are given by equation (E3.10):

\[
C_{p3}(h) = \pm 0.69 \left( \frac{6.8}{(h-0.1)^2} \right)
\]  

(E3.10)

G 3.6.6 The \( k_3 \) values are the same as for the canopies with back walls.

Figure G 3  Pressure loads on a horizontal structure adjacent to the track

G 3.6.7 For canopies with \( h \geq 7.8 \) m, with or without back walls, \( C_{p3} = 0 \).
G 3.6.8 For platform canopies lower than 4 m above rail level, or canopies with a back wall closer than 3.45 m from track centreline, special studies should be undertaken to evaluate the pressure values.

G 3.7 **Trestle platforms**

G 3.7.1 Trestle platform structures at GB platform height (915 mm) above rail level belong to this category.

G 3.7.2 The true pressure field around the train is considered to be represented by the equivalent distributed pressures \( +p_{4k} \) and \( -p_{4k} \), acting vertically on the surface of the platform, each of which is 5 m long and moving at the speed of the train. These pressures are a maximum at the edge of the platform closest to the running track and reduce with increasing lateral distance.

G 3.7.3 The equivalent pressures for trestle platforms located at \( Y \) from track centre are determined from equation (E3.11):

\[
p_{4k} = 0.5 \rho v^2 k_4 C_{p4}(Y)
\]

(E3.11)

G 3.7.4 The characteristic pressure coefficient values for a trestle platform at a distance \( Y \) from the track centre are given by equation (E3.12):

\[
C_{p4}(Y) = \frac{2.0}{(Y+3.23)^2}
\]

(E3.12)

Where \( k_4 = 1.0 \) for bluff trains, 0.80 for intermediate trains and 0.32 for streamlined trains.

G 3.8 **Effect of wind on aerodynamic actions caused by trains**

G 3.8.1 If the effect of ambient wind has to be included in the estimate of the pressures during train passage, the wind speed component parallel to the track can be added to the train speed in the equations for the equivalent pressures, for example equation (E3.11).

---

**Figure G 4** Definition sketch of the co-ordinate system
G 3.9 Notation

G 3.9.1 The parameters used within equations (E3.1) to (E3.12) and Figure G 4 of this document are defined below:

$C_{pi}$: Non-dimensional characteristic pressure coefficient value, $i=1$ to 4.

$C_{p2,\text{bluff}}$: Non-dimensional characteristic pressure coefficient value for overbridges and bluff trains in equation (E3.6a).

$C_{p2,\text{int_str}}$: Non-dimensional characteristic pressure coefficient value for overbridges and intermediate / streamlined trains in equation (E3.6b) and (E3.6c).

$h$: Distance from top of rail to structure over the railway (overbridge / canopy), (m).

$k$: Parameter that specifies the effect of train type.

$p_{ik}$: Equivalent pressure, $i=1$ to 4, (N·m$^{-2}$).

$v$: Train speed, (m·s$^{-1}$).

$W$: Width of structure over railway (overbridge) in x direction, (m).

$x$: Distance along the track, (m).

$y$: Lateral distance from centre of track, (m).

$Y$: Lateral distance of vertical structures from centre of track, (m).

$z$: Vertical distance from the track, (m).

$\rho$: Density of air = 1.225, (kg·m$^{-3}$).
Part 4  Guidance on the Design of Structures to Resist Derailment Actions

G 4.1 Structures over, or adjacent to, the railway subject to the effects of collision loading

G 4.1.1 Background

G 4.1.1.2 However, there may be circumstances where it is appropriate for all parties to consider an alternative design strategy, other than design for the recommended impact values for Class A and B structures. This part of the document therefore provides guidance on the use of risk assessment to support a decision to design structures for a reduced loading.

G 4.1.1.3 Potentially, impact forces due to a derailed train can be extremely high if the full mass of the train is mobilised at the point of impact at line speed. However, for many practical situations, the likelihood of an impact and its magnitude being representative of the worst case scenario (maximum mass impacting a structure at maximum line speed) in the event of a derailment is low.

G 4.1.1.4 BS EN 1991-1-7:2006, clause 4.5.1.2(1) NOTE 1 permits the UK National Annex to define structures to be included within Class A and B. However, the UK National Annex leaves this choice to be ‘agreed for the individual project’. In most cases the choice of structure class will be straightforward. Based on the definitions in BS EN 1991-1-7:2006 Table 4.3, Class A primarily represents occupied buildings and Class B represents bridges and unoccupied buildings. In circumstances where the choice is less straightforward, it is the number of people using the structure and the duration of their stay that are the key distinguishing factors influencing the assignment of a structure class. It is appropriate that such choices are made for a specific project, where the risks are known for particular locations and operating conditions.

G 4.1.1.5 BS EN 1991-1-7:2006, together with its UK National Annex, provides values for impact forces that are intended to achieve a generally acceptable level of robustness for railway structures that are located adjacent to and span above the track, given that the likelihood of an impact and its magnitude being representative of the worst case scenario in the event of a derailment is low.

G 4.1.1.6 Logically the level of robustness should depend on the parameters set out in G 4.1.2.1 of this document. However, it is often unclear what impact scenarios are represented by the values provided in BS EN 1991-1-7:2006 and how they might be changed to reflect the operational conditions at a specific location.

G 4.1.1.7 Therefore, where the application of the values set out in BS EN 1991-1-7:2006 would result in an impractical or uneconomic design, those responsible for the management of railway infrastructure are sometimes faced with the problem of deciding what a reasonable impact force to use in design is.

G 4.1.2 Risk factors
G 4.1.2.1 In principle, wherever structures are adjacent to, or span above, the track they are vulnerable to impact from a derailed train. In practice, the vulnerability of a structure depends on a number of factors including the:

a) Likelihood of a derailment.

b) Proximity of the structure’s supports to the track.

c) Robustness of the supports and the supported structure.

d) Degree of support protection provided.
Guidance on Loading Requirements for the Design of Railway Structures

e) Passing speed of the train.
f) Track alignment and features in the vicinity of the structure.
g) Ground topography adjacent to the track.

G 4.1.2.2 Potential development close to the railway may also be relevant.

G 4.1.2.3 The consequences of a derailment which leads to impact are dependent upon:

a) Occupancy of buildings related to the structure.
b) Number of pedestrians and vehicles (road or rail) using the structure.
c) Occupancy of passing trains.

G 4.1.2.4 Structures that might be vulnerable include:

a) Station buildings adjacent to the track.
b) Other buildings and structures adjacent to the track.
c) Structures, such as bridges or buildings, spanning the track.

G 4.1.2.5 Compliance with the design values provided in BS EN 1991-1-7:2006 has typically been problematic at stations where the structure supports are located close to the track and fall within the 'hazard zone'. In some cases the costs of providing a compliant design have been unreasonably high, compared to the consequences of doing nothing or the implementation of risk mitigation measures. Risk assessment has been used to support a decision to use reduced loading and to demonstrate that the risks of impact following derailment have been controlled to an acceptable level (see G 4.3 of this document).

G 4.1.2.6 BS EN 1991-1-7:2006, section 4.5.2 concerns design for 'Structures located in areas beyond track ends'. GI/GN7616 'Guidance on Station Platform Geometry' Appendix A, sets out a methodology for 'Assessment of Overrun Risk Zone Behind Buffer Stop'.

G 4.1.2.7 In many cases, a more holistic view of the risks inherent in the design of the structure and the operation of the railway can be beneficial. For example, speed will be a key factor influencing both the likelihood and consequences of a derailment leading to impact with a railway structure, as well as the magnitude of the impact force.

G 4.1.2.8 Factors that will influence the vulnerability of a structure to impact and the consequences of a derailment at a particular location are listed in clauses G 4.1.2.1 and G 4.1.2.2 of this document. A key consideration for vulnerability at stations is whether the structure support is located within a platform and if so the degree of protection against impact that might be available from the platform construction.

G 4.2 Design to BS EN 1991-1-7:2006

G 4.2.1 Design requirements

G 4.2.1.1 BS EN 1991-2:2003, clause 6.7.2 requires design for collision following derailment to be undertaken in accordance with BS EN 1991-1-7:2006. This is because derailment loads leading to impact with structures over or adjacent to the track are considered to be accidental actions, which are covered in BS EN 1991-1-7:2006.
### Guidance on Loading Requirements for the Design of Railway Structures

#### G 4.2.2 Structure classification

##### G 4.2.2.1

The requirements for design are dependent upon the structure classification. The structure classification system used in BS EN 1991-1-7:2006, Table 4.3, is based upon the use and occupancy characteristics of the affected structure. In practice, the speed of passing traffic and the type of traffic can influence the likelihood of derailment and the potential consequences following derailment.

##### G 4.2.2.2

The classification system coverage may be summarised as follows:

- **a)** Class A - occupied buildings; clause 4.5.1.4(1) restricts the scope of the design values in BS EN 1991-1-7:2006 Table 4.4 to impact arising from trains with a maximum speed up to 75 mph (120 km/h).

- **b)** Class B - unoccupied buildings and bridges; BS EN 1991-1-7:2006 does not define a maximum train speed but UIC 777-2R recommends that Class B structures apply where passenger trains travel at speeds up to 190 mph (300 km/h) and where freight trains travel at speeds up to 100 mph (160 km/h); design impact forces are given in NA.2.30 of the UK NA to BS EN 1991-1-7:2006.

#### G 4.2.3 Class A structure requirements

##### G 4.2.3.1

Design forces (F<sub>d</sub>x and F<sub>d</sub>y) for Class A structures are provided in BS EN 1991-1-7:2006 Table 4.4. The design forces are intended for ‘continuous walls and wall type structures’ within a distance ‘d’ of 3 m to 5 m from the centreline of the track. Industry practice within GB has been to consider the design forces to be applicable to any structure within the ‘hazard zone’.

##### G 4.2.3.2

BS EN 1991-1-7:2006 permits alternative values for F<sub>d</sub>x and F<sub>d</sub>y to be used for Class A structures where the line speed is ≤ 30 mph (50 km/h) (BS EN 1991-1-7:2006, clause 4.5.1.4(4) NOTE) and also where it is > 75 mph (120 km/h) (BS EN 1991-1-7:2006, clause 4.5.1.4(5) NOTE).

##### G 4.2.3.3

For speeds ≤ 30 mph (50 km/h), the alternative design values for Class B structures in the UK National Annex for BS EN 1991-1-7:2006, clause NA.2.30 can be used. Where it is not practicable to design a structure to resist even the reduced Class B values, then the provision of mitigation measures or a structure-specific risk assessment may be considered.

##### G 4.2.3.4

For speeds >75 mph (120 km/h), a structure-specific risk assessment can be undertaken to support the use of mitigation measures.

##### G 4.2.3.5

For Class A or B structures with potentially high consequences from impact (for example, individual columns located within the ‘hazard zone’), consideration can be given to designing the span over the railway to incorporate sufficient continuity such that the loss of any one column will not lead to the collapse of the remainder of the structure under the permanent loads and accompanying variable actions.

#### G 4.2.4 Class B structure requirements

##### G 4.2.4.1

Within the classification for Class B structures, it is possible that a bridge may be vulnerable to impact where line speeds are low, such as at a station. This is a common situation within GB and is a relevant factor when determining a reduced value of the design impact force provided in the UK National Annex to BS EN 1991-1-7:2006.

##### G 4.2.4.2

The level of occupancy of a structure, the characteristics of the track and the adjacent topography, and the speed of the traffic are primary factors which influence the risk at a particular site. Although buildings are likely to have high occupancy compared to bridges, it is also possible that footbridges spanning railways may be subject to high pedestrian flows, and road bridges over railways can be occupied by stationary traffic, at particular times of the day on a regular basis. These conditions can arise in situations where the line speed for railway traffic below is high. The risks at a particular site are best determined for the specific project where the risks are better able to be assessed.
G 4.2.4.3 For Class B structures in particular, it is recognised that it is not economically possible to design for the potential maximum force that can arise due to train impact at high speed. The potential magnitude of the impact force is very large, as was the case at Eschede where the bridge was demolished with a resulting very high rate of casualties. BS EN 1991-1-7:2006 recognises this and consequently permits the use of risk assessment for determination of appropriate design forces.

G 4.2.4.4 The GB approach for Class B structures within the National Annex to BS EN 1991-1-7:2006, is to provide minimum robustness requirements for columns in the ‘hazard zone’, and additionally to allow for redundancy of bridge columns located within the ‘hazard zone’ (see G 4.2.5 of this document).

G 4.2.5 Hazard zone

G 4.2.5.1 The concept of a ‘hazard zone’ is not explicitly covered in BS EN 1991-1-7:2006, although BS EN 1991-1-7:2006, Table 4.4 specifies that the design impact forces are 0 kN for distances of ‘d’ greater than 5.0 m. It may therefore be assumed that the risk of impact between a structure and a derailed train at this distance from the track centreline is acceptably low. The UK National Annex for BS EN 1991-1-7:2006, clause NA.2.30 defines the ‘hazard zone’ as an area extending for a distance of 4.5 m from the cess rail (and anywhere between the tracks). For a GB gauge track (1.435 m), this represents a zone more than approximately 5.2 m from the track centreline where the risk from impact is low. For consistency with the Eurocode requirements and the established GB practice, it is recommended that the GB value for ‘d’, beyond which design for impact is not required, be taken as 5.2 m.

G 4.2.5.2 Where there is a high risk that a derailed train may go significantly beyond the operating envelope, due to the presence of switches and crossings (S&C) and horizontal curvature, for example, the ‘hazard zone’ may not be a reliable indicator of risk, and a site-specific risk assessment would be beneficial.

G 4.2.5.3 If impact does occur, the structural characteristics of the impacted structure will have a significant influence on the magnitude of the applied force. Account can be taken of this in analysis which considers the dynamic and non-linear behaviour of the structure and the train, but this aspect is not considered further within this document.

G 4.3 Risk assessment

G 4.3.1 Commission Regulation (EC) No. 352/2009 established a ‘common safety method on risk evaluation and assessment’ (the CSM RA). The CSM RA, contained in Annex I to the regulation, sets out a mandatory risk management process for the rail industry that is common across Europe. The CSM RA has applied to all significant changes to the railway system since 01 July 2012. The changes may be of a technical (engineering), operational or organisational nature (where the organisational changes could have an impact on the operation of the railway). The CSM RA also applies if a risk assessment is required by a technical specification for interoperability (TSI); and is used to ensure safe integration of a structural subsystem into an existing system in the context of an authorisation for placing in service in accordance with the Railway Interoperability Directive 2008/57/EC.

G 4.3.2 Commission Implementing Regulation (EU) No 402/2013 establishes a revised common safety method for risk evaluation and assessment. The revised CSM RA has been in force since 23 May 2013 (meaning it can be used from that date), and will apply from 21 May 2015 (meaning that it must be used from that date), at which time Commission Regulation (EC) No. 352/2009 is repealed.
The Office of Rail Regulation issued a policy statement on the relationship between the CSM RA and other risk assessment requirements (RGD-2013-06, December 2013). This states:

Where a change is not significant [that is, the change is not judged to meet the criteria for significance set out in CSM RA], it will fall to the proposer of the change to consider domestic legislative requirements ... which require a suitable and sufficient risk assessment to be undertaken. It is possible to adopt the CSM RA approach even when there is no legal requirement to do so (for example, when a change is not significant). Following the CSM approach correctly in these circumstances is likely to mean that domestic safety legislation is complied with.

It is the responsibility of the proposer of a change to identify the hazards associated with the proposed change and to determine what safety measures are needed to control the risks to an acceptable level. CSM RA sets out that the risk acceptability of a significant change should be evaluated by using one or more of three risk acceptance principles: the application of codes of practice, a comparison with similar [reference] systems, and explicit risk estimation.

When 'Application of codes of practice' has been selected as a risk acceptance principle, the CSM RA, clause 2.3.1 states that 'the proposer, with the support of other involved actors ... shall analyse whether one or several hazards are appropriately covered by the application of relevant codes of practice'.

BS EN 1991-1-7:2006 and its UK National Annex can be considered to be a relevant code of practice when evaluating hazards associated with impact loads on structures over or adjacent to the railway.

It is possible that a structure located within the hazard zone cannot be designed to control the risk due to impact to an acceptable level using the impact loading for a Class A or Class B structure set out in BS EN 1991-1-7:2006 and its UK National Annex. It is also possible that the impact loads specified in BS EN 1991-1-7:2006 could result in an impractical or uneconomic design, taking into account existing or additional preventative and / or protective measures. In such cases and in cases where BS EN 1991-1-7:2006 and its UK National Annex do not set out a suitable impact load or where an individual project is permitted to identify a suitable impact load, it may be necessary to decide on a reasonable alternative impact load to use as a basis for design.

To identify a reasonable alternative which controls the risks to an acceptable level, the risk acceptance principles of comparison with similar [reference] systems, and / or explicit risk estimation can be used.

Clause G 4.1.2.1 of this document sets out factors that can affect the vulnerability of structures which are adjacent to, or span above, the track which can be used to estimate the risk.

The RSSB Safety Risk Model (SRM) provides annual derailment risk data for the GB mainline network. Further information may be found from the Risk Profile Bulletin which is available from the RSSB website.

UIC 777-2R Appendix F provides a methodology for the estimation of the risk associated with the hazard of train derailment affecting structures over or adjacent to the railway. However, it is important to note that the UIC 777-2R methodology does not consider all the factors set out in clause G 4.1.2.1 of this document (for example, ground topography adjacent to the track) for estimation of the risk.
G 4.3.12 The UIC methodology requires important parameters to be used for determination of the likelihood of train derailment. These are the derailment rate for trains per train km (er) and the number of trains per day (Zd). GB specific values for these parameters may be determined from the derailment risk data for the GB mainline network which may be found from the Risk Profile Bulletin (www.rssb.co.uk/SPR/Pages/SAFETYRISKMODEL.aspx), and the current / proposed frequency of service of the relevant train operating company (TOC).

G 4.3.13 UIC777-2R provides two pre-determined values of derailment rate for passenger trains and freight trains, with and without S&C on the approach to a bridge. The derailment rate for the case where S&C are present is assumed to be a factor of 10 greater than where S&C is not present.

G 4.3.14 Appendix B of this document sets out guidance on the application of the UIC 777-2R risk assessment methodology to GB structures.

G 4.3.15 Further more detailed guidance on the application of CSM RA is provided in a suite of six complementary GNs – GE/GN8640, GE/GN8641, GE/GN8642, GE/GN8643, GE/GN8644 and GE/GN8645.
Appendix A  Guidance on the Infrastructure Technical Specification for Interoperability

G A.1  Introduction
G A.1.1  The guidance provided in this Appendix relates to the requirements for structures proposed for inclusion in the revised INF TSI, based on the latest available draft at the time of writing (version 4.5, 23/04/2013). This guidance will be withdrawn when guidance note GI/GN7608 is next updated to include the revised INF TSI.

G A.2  Structures resistance of new bridges to traffic loads
G A.2.1  Vertical loads

Extract from the INF TSI [draft]
4.2.7.1.1. VERTICAL LOADS

(1) Structures shall be designed to support vertical loads in accordance with the following load models, defined in EN 1991-2:2003/AC:2010:

(a) Load Model 71, as set out in EN 1991-2:2003/AC:2010 paragraph 6.3.2 (2)P.

(b) In addition, for continuous bridges, Load Model SW/0, as set out in EN 1991-2:2003/AC:2010 paragraph 6.3.3 (3)P.

(2) The load models shall be multiplied by the factor alpha (α) as set out in EN 1991-2:2003/AC:2010 paragraphs 6.3.2 (3)P and 6.3.3 (5)P.

(3) The value of factor alpha (α) shall be equal to or greater than the values set out in Table 11.

Table 11 : Factor alpha(α) for the design of new structures – Not extracted.

G A.2.1.1  The load classification factor alpha (α) can be used to vary the magnitude (+ve or -ve) of railway traffic loading to suit the capacity required for a particular route. BS EN 1991-2:2003, clause 6.3.2(3) NOTE permits the value to be specified in the UK National Annex. The UK National Annex for BS EN 1991-2:2003, clause NA.2.48 recommends a value for α of 1.1 for compatibility with pre-Eurocode safety levels. However, higher values may be appropriate for a particular project, where the impact on safety and economy over the structure’s life can be taken into account.

G A.2.2  Allowance for dynamic effects of vertical loads

Extract from the INF TSI [draft]
4.2.7.1.2. ALLOWANCE FOR DYNAMIC EFFECTS OF VERTICAL LOADS

(1) The load effects from the Load Model 71 and Load Model SW/0 shall be enhanced by the dynamic factor phi (Φ) as set out in EN 1991-2:2003/AC:2010 paragraphs 6.4.3 (1)P and 6.4.5.2 (2).

(2) For bridges for speeds over 200 km/h where EN 1991-2:2003/AC:2010 paragraph 6.4.4 requires a dynamic analysis to be carried out the structure shall additionally be designed for HSLM defined in EN 1991-2:2003/AC:2010 paragraphs 6.4.6.1.1 (3) to (6) inclusive.

(3) It is permissible to design new bridges such that they will also accommodate an individual passenger train with higher axle loads than covered by HSLM. The dynamic analysis shall be undertaken using the characteristic value of the loading from the individual train taken as the design mass under normal payload in accordance with Annex K with an allowance for passengers in standing areas in accordance with Note 1 of Annex K.
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G A.2.2.1 The requirements for making allowance for dynamic effects in the INF TSI, clause 4.2.7.1.2 (1), are generally appropriate for bridges which carry rail traffic at speeds up to and including 125 mph (200 km/h). However, for dynamically sensitive bridges the UK NA for BS EN 1991-2:2003 makes provision for undertaking a dynamic analysis where railway traffic speeds are less than 125 mph (200 km/h). For bridges carrying rail traffic operating at speeds in excess of 125 mph (200 km/h), a dynamic analysis in accordance with the INF TSI, clause 4.2.7.1.2 (2) is required.

G A.2.3 Design track twist due to rail traffic actions

Extract from the INS TSI [draft]
4.2.7.1.6. DESIGN TRACK TWIST DUE TO RAIL TRAFFIC ACTIONS

(1) The maximum total design track twist due to rail traffic actions shall not exceed the values set out in paragraph A2.4.4.2.2(3)P in Annex A2 to EN 1990:2002 issued as EN 1990:2002/A1:2005.

G A.2.3.1 The limit for the maximum total design track twist set out in the INF TSI, clause 4.2.7.1.6, is taken into account when determining the components of twist due to vertical track alignment, track alignment defects, and deformation of the track due to rail traffic load. Requirements for the steepest permitted designed cant gradient and repair of track twist are set out in GC/RT5021.

G A.2.4 Horizontal loads from rail traffic

Extract from the INF TSI [draft]
4.2.7.1.4. NOSING FORCES


Extract from the INF TSI [draft]
4.2.7.1.3. CENTRIFUGAL FORCES

(1) Where the track on a bridge is curved over the whole or part of the length of the bridge, the centrifugal force shall be taken into account in the design of structures as set out in EN 1991-2:2003/AC:2010 paragraphs 6.5.1 (2), (4)P and (7).

Extract from the INF TSI [draft]
4.2.7.1.5. ACTIONS DUE TO TRACTION AND BRAKING (LONGITUDINAL LOADS)

(1) Traction and braking forces shall be taken into account in the design of structures as set out in EN 1991-2:2003/AC:2010 paragraphs 6.5.3 (2)P, (4), (5), (6), and (7)P.

G A.2.4.1 No GB guidance is associated with the above.
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G A.2.5  Equivalent vertical loading for new earthworks and earth pressure effects

<table>
<thead>
<tr>
<th>Extract from the INF TSI [draft]</th>
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<tbody>
<tr>
<td><strong>4.2.7.2. EQUIVALENT VERTICAL LOADING FOR NEW EARTHWORKS AND EARTH PRESSURE EFFECTS</strong></td>
</tr>
<tr>
<td>(1) Earthworks shall be designed and earth pressure effects shall be determined taking into account the vertical loads produced by the Load Model 71, as set out in EN 1991-2:2003/AC:2010 paragraph 6.3.2 (2).</td>
</tr>
<tr>
<td>(2) The equivalent vertical loading shall be multiplied by the factor alpha (α) as set out in EN 1991-2:2003/AC:2010 paragraph 6.3.2 (3)P. The value of α shall be equal to or greater than the values set out in Table 11.</td>
</tr>
</tbody>
</table>

G A.2.5.1 The INF TSI section 4.2.7.2 covers not only the vertical load effects on earthworks but also the consequential additional horizontal earth pressure effects due to the weight of trains acting on the backfill (surcharge). These vertical and horizontal loads on earthworks are utilised in the design of bridge abutments, and similar earth retaining elements (for example, retaining walls and wing walls).

G A.3 Design of structures for aerodynamic actions

G A.3.1 Structures subject to the aerodynamic effects of passing trains

<table>
<thead>
<tr>
<th>Extract from the INF TSI [draft]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4.2.7.3 RESISTANCE OF NEW STRUCTURES OVER OR ADJACENT TO TRACKS</strong></td>
</tr>
</tbody>
</table>

G A.3.1.1 BS EN 1991-2:2003, clause 6.6.1(3) NOTE permits alternative values for aerodynamic actions from passing trains to be specified. These are included within Part 3 of this document.
### Appendix B  Application of the UIC 777-2R Risk Assessment Methodology to GB Structures

**G B.1  UIC 777-2R risk assessment methodology**

**G B.1.1  The risk assessment methodology is based on that included within Appendix F of UIC 777-2R.**

**G B.1.1.1  The methodology involves several defined steps which provide the values of the parameters that can be used in an explicit risk assessment. These steps are summarised in Table B1 for each risk scenario.**

<table>
<thead>
<tr>
<th>Risk scenario</th>
<th>Parameters for risk assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Likelihood of a train becoming derailed on the approach to a bridge.</td>
<td>The likelihood (P1) is determined from the following equation: P1 = e_r x d x Z_d x 365 x 10^{-3}. Where: e_r = the derailment rate for trains per train km. d = longest derailment path in metres = V^2/80 (assuming a deceleration of 3 m/s^2 and a derailment path parallel to the track). See Figure G 5. Z_d = the number of trains per day.</td>
</tr>
<tr>
<td>Likelihood of the train colliding with the bridge.</td>
<td>The likelihood (P2) is determined from the following equations: P2 = [(b-a)/b]^2 x 0.5 x c/d – for a single track. P2 = [(b-a)/b]^2 + [(b – (a+4.2))/b]^2 x 0.25 x c/d – for a double track. Where the distance parameters below are shown in Figure G 5: d = longest derailment path in metres. V = speed of train in km per hour at point of derailment. b = the predicted maximum lateral deviation in metres of a derailed train = V^{0.95}. a = lateral distance in metres between centreline track and face of structure. c = distance parallel to the tracks at lateral distance ‘a’ exposed to the risk of impact from a derailed train. c = (d/b) x (b-a) ((for values of b &lt; a c may be taken as zero).</td>
</tr>
<tr>
<td>Likelihood of the bridge collapsing as a result of the impact.</td>
<td>The likelihood (P3) is determined from the following equation: P3 = [1 \rightarrow 2/3{[2(2b - 2a - t) / (b - a)^2]} x \alpha. ] For: b – t – a &gt; 0. Where: t = (a x d') / (d – d').</td>
</tr>
</tbody>
</table>
**Guidance on Loading Requirements for the Design of Railway Structures**

\[ t = \text{the lateral deviation over which the remaining speed of the derailed train has fallen below 40 mph (60 km/h).} \]

\[ d' = \text{the longitudinal distance of the longest derailment path (parallel to the track) over which the remaining speed of the train has fallen below 40 mph (60 km/h). A constant 45 m may be assumed based on a constant deceleration of 3 m/s}^2. \]

\[ \alpha = \text{a dimensionless factor to take account of support robustness and structural continuity (for } \alpha = 1 \text{ it is assumed that all impacts with the support result in collapse for speeds greater than or equal to 40 mph (60 km/h)).} \]

<table>
<thead>
<tr>
<th>Likelihood of a train travelling on another track colliding with the derailed train.</th>
<th>The likelihood (P4) = 0.1 (where there are two or more tracks under the bridge). This value may be increased where, the line carries &gt; 100 trains per day, trains using the line are restricted to a narrow speed band (for example, High-speed passenger traffic or high-volume freight traffic).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of train travelling in the opposite direction.</td>
<td>Assume either a passenger train or a freight train.</td>
</tr>
<tr>
<td>Likelihood of the resulting scenarios.</td>
<td>The likelihood (probability - ( P_{se} )) of risk for each risk scenario is calculated using a fault-event tree analysis (9 scenarios involving derailment &gt; impact &gt; collapse &gt; secondary collision with passenger or freight trains).</td>
</tr>
</tbody>
</table>

**Table G 2** Risk assessment procedure

**Figure G 5** UIC 777-2R dimensions

G B.1.1.2 Figure G 5 of this document indicates the dimensions that are used within the UIC 777-2R methodology for determination of the required risk parameters.
G B.1.1.3 For the GB mainline railway network annual derailment risk data is available from [www.safetyriskmodel.co.uk](http://www.safetyriskmodel.co.uk). The derailment rate is an average value across the whole network and does not take account of local risk factors that may make the derailment rate higher or lower than the average value at a specific site (for example, forward facing S&C).

G B.1.1.4 The likelihood (probability) of each outcome for the resulting risk scenarios is then used to calculate the annual risk ($D_{szi}$) from the expression:

$$D_{szi} = P_{szi} \times S_{zi}.$$  

Where $P_{szi} = P1 \times P2 \times P3 \times P4 \times P5$, and is the probability of a particular combination of events (derailment, impact, collapse, secondary collision with another train) occurring for each potential scenario ($S_{zi}$ for $i = 1$ to $9$).

And

$S_{zi}$ is the number of fatalities assumed for each scenario.

G B.1.1.5 The annual cost ($R_{bpm}$) associated with preventing the fatalities from occurring (the benefits) for each risk scenario, is then calculated from the expression:

$$R_{bpm} = \sum R_{bpi} = V_{pf} \times A_{pf} \times D_{szi}.$$  

Where:

$V_{pf}$ is the value for preventing a fatality (available from [www.RSSB.co.uk](http://www.RSSB.co.uk)).

$A_{pf}$ is the aversion factor applied to the assumed number of fatalities. It is assumed to be 1.0 except where account is explicitly taken of additional factors such as passenger loading, crash worthiness of vehicles, average speed and occupancy of adjacent buildings. Mean fatality numbers are taken from the Risk Profile Bulletin ([www.safetyriskmodel.co.uk](http://www.safetyriskmodel.co.uk)).

G B.1.1.6 The theoretical annual discounted cost of providing preventative measures or for the provision of protective measures, is then calculated from the expression:

$$A = \frac{Cs \times (1 + Z)^N}{(1 + Z)^N - 1}.$$  

Where $Cs = \text{investment cost of the protective measure} = \sum(A) / (1 + Z)^N$.

$N = \text{the expected design life (years)}$.

$Z = \text{average annual rate of interest}$.

G B.1.1.7 The total annual cost ($C_{tot}$) is then calculated from the following expression:

$$C_{tot} = A + (Y \times Cs).$$  

where $Y = \text{average annual maintenance costs assumed to be a proportion of the investment cost (Cs)}$. The value for annual maintenance costs should be estimated for the individual project.

G B.1.1.8 The effectiveness of the measure is then obtained from a comparison of the ratio of the annual cost of preventing fatalities ($R_{bpm}$) to the total annual cost of providing preventative or protective works ($C_{tot}$). Where the ratio $R_{bpm} / C_{tot}$ is equal to or less than one, the measures may be considered to be appropriate. This is dependent upon the level of uncertainty in the parameters used for determination of the benefits and costs and whether the cost of the safety benefits is considered to be ‘grossly disproportionate’ to the safety benefits. This is determined by professional judgement, paying particular attention to the degree of uncertainty in the assessment of costs and safety benefits. Case law has established that a safety measure is reasonably practicable unless the cost is ‘grossly disproportionate’.
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G B.2 Example applications of the risk assessment methodology

G B.2.1 Scenario one – footbridge support located within platform – minimal protection works required

G B.2.1.1 Derailment of a passenger train on a double track section within a station area (with switches and crossings located on the approach to the station).

G B.2.1.2 Passing and stopping trains use the track adjacent to the platform.

G B.2.1.3 Assumptions:

- a) Train passing speed, \( V = 50 \text{ mph} \) (80 km/h).
- b) Number of trains per day, \( Z_d = 300 \).
- c) Derailment rate, \( e_r = 2.5 \times 10^{-08} \).
- d) Robustness factor, \( \alpha = 1.0 \) (except where the likelihood of collapse is reduced).
- e) Distance between centreline track and nearest support face, \( a = 2.0 \text{ m} \).

G B.2.1.4 Using these values, the risk parameters are calculated in accordance with the equations provided in Table G 2.

<table>
<thead>
<tr>
<th>Risk assessment parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
</tr>
<tr>
<td>11.14</td>
</tr>
</tbody>
</table>

Table G 3 Risk assessment parameters

G B.2.2 Risk of fatalities

G B.2.2.1 The probability that each risk scenario is achieved is calculated using the equations and values for each probability included within Table G 2 (see Table G 3 for values of the risk assessment parameters).
### Guidance on Loading Requirements for the Design of Railway Structures

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Derailment</th>
<th>Impact bridge</th>
<th>Bridge collapse</th>
<th>Secondary collision</th>
<th>Passenger train</th>
<th>Freight train</th>
<th>Probability for each scenario ($P_{szi}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.19E-04</td>
<td>0.276078</td>
<td>0.131188</td>
<td>0.2</td>
<td>0.5</td>
<td>7.93E-07</td>
<td>derailment&gt;impact&gt;collapse&gt;secondary collision PT.</td>
</tr>
<tr>
<td>2</td>
<td>2.19E-04</td>
<td>0.276078</td>
<td>0.131188</td>
<td>0.2</td>
<td>0.5</td>
<td>7.93E-06</td>
<td>derailment&gt;impact&gt;collapse&gt;secondary collision FT.</td>
</tr>
<tr>
<td>3</td>
<td>2.19E-04</td>
<td>0.276078</td>
<td>0.131188</td>
<td>0</td>
<td></td>
<td>7.93E-06</td>
<td>derailment&gt;impact&gt;collapse.</td>
</tr>
<tr>
<td>4</td>
<td>2.19E-04</td>
<td>0.276078</td>
<td>0</td>
<td>0.2</td>
<td>0.5</td>
<td>6.05E-06</td>
<td>derailment&gt;impact&gt;secondary collision PT.</td>
</tr>
<tr>
<td>5</td>
<td>2.19E-04</td>
<td>0.276078</td>
<td>0</td>
<td>0.2</td>
<td>0.5</td>
<td>6.05E-06</td>
<td>derailment&gt;impact&gt;secondary collision FT.</td>
</tr>
<tr>
<td>6</td>
<td>2.19E-04</td>
<td>0.276078</td>
<td>0</td>
<td>0</td>
<td></td>
<td>6.05E-05</td>
<td>derailment&gt;impact.</td>
</tr>
<tr>
<td>7</td>
<td>2.19E-04</td>
<td>0</td>
<td>0</td>
<td>0.2</td>
<td>0.5</td>
<td>2.19E-05</td>
<td>derailment&gt;secondary collision PT.</td>
</tr>
<tr>
<td>8</td>
<td>2.19E-04</td>
<td>0</td>
<td>0</td>
<td>0.2</td>
<td>0.5</td>
<td>2.19E-05</td>
<td>derailment&gt;secondary collision FT.</td>
</tr>
<tr>
<td>9</td>
<td>2.19E-04</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td>2.19E-04</td>
<td>derailment.</td>
</tr>
</tbody>
</table>

**Combined probability for all scenarios $\Sigma P_{szi}$**

3.45E-04

---

**Table G 4** Combined probability for all scenarios

**G B.2.3** Value for prevention of fatalities

**G B.2.3.1** The number of fatalities for each scenario at a speed of 50 mph (80 km/h) is taken from Table 7 of UIC777-2R. Values are only included within Table 7 for speeds as low as 75 mph (120 km/h) and this figure has been assumed to apply to 50 mph (80 km/h) also.

**G B.2.3.2** For passenger trains the number of fatalities (or equivalent fatalities) for each scenario $S_{zi}$ is included within Table G 5.

**G B.2.3.3** The equation for annual risk from each scenario $D_{szi}$ is included in G B.1.1.4.
G B.2.3.4 The value for the aversion factor $A_{pf} = 2.0$ (see G B.1.1.5 of this document) to allow for high passenger footfall at the station and the frequency of fully loaded passing trains.

G B.2.3.5 The annual perceived risk from each scenario is calculated from the annual risk and the aversion factor $R_b = A_{pf} \times D_{sz}$. 

G B.2.3.6 The total annual perceived risk from derailment of passenger and freight trains (where necessary) is calculated from the sum of the risk for each scenario:

$$\sum R_{bp} = R_{b1} + R_{b2} + R_{b3} + R_{b4} + R_{b5} + R_{b6} + R_{b7} + R_{b8} + R_{b9}.$$  

$$\sum R_{bf} = R_{b10} + R_{b11} + R_{b12} + R_{b13} + R_{b14} + R_{b15} + R_{b16} + R_{b17} + R_{b18}.$$  

G B.2.3.7 The value placed on preventing a human fatality $V_{pf}$ is based on the latest value from www.RSSB.co.uk (see Table G 5).

G B.2.3.8 The value of preventing the number of fatalities expected from the total annual perceived risk for passenger and freight trains is:

$$R_{bpm} = V_{pf} \times R_{bp}.$$  

$$R_{bpf} = V_{pf} \times R_{bf}.$$  

<table>
<thead>
<tr>
<th>Scenario</th>
<th>$S_{sz}$</th>
<th>$D_{sz}$</th>
<th>$A_{pf}$</th>
<th>$R_{bp}$</th>
<th>$V_{pf}$</th>
<th>$R_{bpmi}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
<td>1.19E-05</td>
<td>2</td>
<td>2.38E-05</td>
<td></td>
<td>derailment&gt;impact&gt;collapse&gt;secondary collision PT.</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>7.93E-06</td>
<td>2</td>
<td>1.59E-05</td>
<td></td>
<td>derailment&gt;impact&gt;collapse&gt;secondary collision FT.</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>7.93E-05</td>
<td>2</td>
<td>1.27E-04</td>
<td></td>
<td>derailment&gt;impact&gt;collapse.</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>6.65E-05</td>
<td>2</td>
<td>1.33E-04</td>
<td></td>
<td>derailment&gt;impact&gt;secondary collision PT.</td>
</tr>
<tr>
<td>5</td>
<td>7</td>
<td>4.23E-05</td>
<td>2</td>
<td>8.46E-05</td>
<td></td>
<td>derailment&gt;impact&gt;secondary collision FT.</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>3.02E-04</td>
<td>2</td>
<td>6.05E-04</td>
<td></td>
<td>derailment&gt;impact.</td>
</tr>
<tr>
<td>7</td>
<td>9</td>
<td>1.97E-04</td>
<td>2</td>
<td>3.94E-04</td>
<td></td>
<td>derailment&gt;secondary collision PT.</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>1.31E-04</td>
<td>2</td>
<td>2.63E-04</td>
<td></td>
<td>derailment&gt;secondary collision FT.</td>
</tr>
<tr>
<td>9</td>
<td>3</td>
<td>6.57E-04</td>
<td>2</td>
<td>1.31E-03</td>
<td></td>
<td>Derailment.</td>
</tr>
<tr>
<td><strong>Total value for preventing fatalities $\sum R_{bpmi}$</strong></td>
<td></td>
<td></td>
<td></td>
<td>2.96E-03</td>
<td>1.76E+06</td>
<td><strong>3.21E+03</strong></td>
</tr>
</tbody>
</table>

**Table G 5** Value for prevention of fatalities
G B.2.4 Cost of preventative and protective measures

G B.2.4.1 The equations for calculation of the average annual cost associated with the provision of preventative or protective measures $A$ and the investment cost of the protective measures $C_s$ is given in clause G B.1.1.6.

G B.2.4.2 The total annual cost of the measure including annual maintenance costs $C_{tot}$ is calculated from the equation in clause G B.1.1.7.

G B.2.4.3 The average annual maintenance cost is assumed to be 2% of the investment cost $C_s$.

G B.2.4.4 The investment cost is usually based on an estimated cost of works and future maintenance at the design stage. This can be based on the cost of similar works, or a specific estimate using a source of Engineering Construction costs such as SPONs.

G B.2.4.5 For the purpose of this example the investment cost of the protective measure $C_s$ has been assumed to be a hypothetical value for the purposes of illustrating the influence of a specific value on the outcome of the benefit: cost ratio.

G B.2.4.6 For scenario one, it is assumed that only minor works are required to isolate the columns from the surrounding platform and that otherwise the platform is sufficiently robust to resist impact forces. The specific assumptions are:

$C_s = 50,000$ pounds.

Expected design life, $N = 125$ years and 30 years.

The annual average interest rate, $Z = 2.5\%$

G B.2.4.7 The calculation results are summarised in Table G 6. For scenario one a positive benefit: cost ratio ($R_{bpm}/C_{tot}$) of 2.2511 is obtained for a 120 year design life and 1.5398 for a 30 year design life. The longer period over which the safety measures have to be funded increases the annual cost of the measures which decreases the benefit: cost ratio.

<table>
<thead>
<tr>
<th>$R_{bpm}$ (£)</th>
<th>$C_s$ (£)</th>
<th>$A$ (£)</th>
<th>$C_{tot}$ (£)</th>
<th>Benefit:Cost ratio $R_{bpm}/C_{tot}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>120</td>
<td>30</td>
<td>120</td>
</tr>
<tr>
<td>5,220</td>
<td>50,000</td>
<td>1318</td>
<td>2389</td>
<td>2318</td>
</tr>
<tr>
<td>6880</td>
<td>175,000</td>
<td>4613</td>
<td>8361</td>
<td>8113</td>
</tr>
<tr>
<td>4100</td>
<td>1,500,000</td>
<td>39543</td>
<td>71666</td>
<td>69543</td>
</tr>
</tbody>
</table>

Table G 6 Benefit: cost ratios

G B.2.4.8 Where the parameters are assumed to take different values for scenarios two and three, different conclusions can be possible.

G B.2.4.9 For scenario two it is assumed that the platform offers no protection in the event of impact and that more significant works are needed to provide protection. The benefits will increase as there is more likelihood that the columns will collapse owing to the inadequate level of protection provided by the platform. The increased cost of the protection works reduces the benefit: cost ratio below 1.0, although only to 0.8483. However, when an allowance is made for uncertainty, this value exceeds 1.0 which may be considered to justify the need for the protection works.
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G B.2.4.10 For scenario three, it is assumed that a much more expensive bridge is provided, without the need for bridge supports within the platform areas and where the supports are more than 5 m from the track centreline. The benefits are reduced owing to the increased clearance which decreases the likelihood of fatalities, and the costs of protection works (assumed to be the additional bridge costs to achieve a 6 m clearance) are greatly increased. The benefit: cost ratio is way below 1.0 in this case and even allowing for uncertainty, it is difficult to justify the additional cost of the new bridge.
Definitions

Abutment
An end support of a bridge whose function is to support the weight of the bridge, the loads exerted by railway vehicles and the earth pressure from the embankment behind the support.

Accidental action
An action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life.

For the purpose of this document the term is generally taken to refer to the effects of derailment of a railway vehicle on, below or adjacent to a railway structure.

Action
For the purpose of this document, a set of rail traffic forces (loads) applied to the structure.

Aerodynamic action
An action that results from the force exerted on railway structures over or adjacent to the railway due to the transient pressures generated by passing railway vehicles.

Bridge
A structure of one or more spans greater than or equal to 1800 mm, whose prime purpose is usually to carry traffic or services over an obstruction or gap.

Bluff
Termed ‘Unfavourable aerodynamic shape’ in BS EN 1991-2:2003; this is the base case for which k = 1.0. A typical example is the Class 66 locomotive.

Culvert
A structure with a span or diameter greater than 0.45 m but less than 1.8 m whose prime purpose is usually to permit water or services to pass under or adjacent to a railway, road or other infrastructure.

Deformation
All deflection and rotational movements in a railway structure due to the effects of railway traffic.

Dynamically sensitive railway structures
Dynamically sensitive railway structures such as bridges, subject to railway traffic, tend to be those which approach or fall outside the upper limit of deflection as determined in accordance with UIC 776-3R Figure 1. For example, shallow (high span depth ratio) solid deck type bridges (reinforced concrete or filler beam decks) with a high mass per metre may tend towards or exceed the upper limit.

Light all metal floors (cross girders with metal floor plates with low or no longitudinal continuity) of through or half through bridges may also be dynamically sensitive.

Embankment
An earthwork that allows railway lines or access roads to pass over low lying ground, or ground liable to flood, at an acceptable level and gradient.

Fatigue
Failure of structural elements and connections subject to the effects of repeated rail traffic loading.

Hazard zone
An area extending for a distance of 5.2 m perpendicular to the track centreline and anywhere between the tracks.
Horizontal load
The resultant force exerted on a railway structure in the direction of travel (longitudinal) or normal to the direction of travel (transverse) as a consequence of the operating characteristics of the moving railway vehicle and its interaction with the track.

Intermediate
Termed ‘Smooth sided rolling stock’ in BS EN 1991-2:2003; the k values are derived from the test results for each structure type, a typical example is the Class 158 leading vehicle.

Lineside structures
For the purpose of this document, structures adjacent to the railway which may be subject to aerodynamic or accidental actions, such as platform canopies and over-line bridge supports.

Mainline railway
Mainline railway has the meaning given to it in the Railways and Other Guided Transport Systems (Safety) Regulations 2006 but excluding any railway in Northern Ireland; the dedicated high speed railway between London St Pancras International Station and the Channel Tunnel; and the Channel Tunnel.

New bridge
For the purpose of this document the term ‘new bridge’ includes total superstructure replacement.

Over-line bridge
A bridge structure of one or more spans which passes over the railway.

Railway structure
A structure below, over or adjacent to the railway which is subject to loading from rail traffic.

For the purpose of this document, the term includes under-line bridges, over-line bridges, lineside structures, tunnel inverted, culverts, buried structures, retaining walls and embankments, which are subject to the vertical and/or horizontal effects of rail traffic.

Retaining wall
For the purpose of this document, an independent wall whose function is to support the pressure from the retained earth behind the wall and additional pressure from rail traffic surcharge loading, where it can affect the wall.

Streamlined
Termed ‘Streamlined rolling stock’ in BS EN 1991-2:2003; the k values are derived from the test results for each structure type, a typical example is the Class 390 leading vehicle.

Trestle
For the purpose of this document, a platform structure comprising a sequence of regularly spaced vertical supports, which are open to air between them, and which support the platform walking surface.

Tunnel invert
The floor constructed close to the base within the tunnel and upon which the railway is supported.

Under-line bridge
A bridge structure of one or more spans which carries the railway over an obstacle, such as a highway for example.
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**Vertical load**
The resultant force exerted on a railway structure in the vertical direction as a consequence of the weight of a railway vehicle, the operating characteristics of the moving railway vehicle, and its interaction with the track.

**Wing wall**
A wall that complements an abutment and whose function is to support the earth pressure from that part of the earth embankment behind the bridge that slopes away from the sides of the track, and additional pressure from rail traffic surcharge loading, where it can affect the wall.
References

The Catalogue of Railway Group Standards give the current issue number and status of documents published by RSSB. This information is also available from www.rgsonline.co.uk.

RGSC 01 Railway Group Standards Code
RGSC 02 Standards Manual

Documents referenced in the text

Technical Specification for Interoperability

HS INF TSI High Speed Infrastructure TSI, Decision 2008/217/EC, (OJ L77/1, 19/03/2008, p1)

Railway Group Standards

GC/RT5021 Track System Requirements
GC/RT5112 Rail Traffic Loading Requirements for the Design of Railway Structures
GE/RT8006 Assessment of Compatibility of Rail Vehicle Weights and Underline Bridges

RSSB documents

GI/GN7608 Guidance on the Conventional Rail and High Speed Infrastructure Technical Specifications for Interoperability
GI/GN7616 Guidance on Interface between Station Platforms, Track and Trains
GE/GN8640 Guidance on Planning an Application of the Common Safety Method on Risk Evaluation and Assessment
GE/GN8641 Guidance on System Definition
GE/GN8642 Guidance on Hazard Identification and Classification
GE/GN8643 Guidance on Risk Evaluation and Risk Acceptance
GE/GN8644 Guidance on Safety Requirements and Hazard Management
GE/GN8645 Guidance on Independent Assessment

Other references

Guidance on Loading Requirements for the Design of Railway Structures

2008 on the interoperability of the rail system within the Community (Recast) (Text with EEA relevance)

776-3R UIC Leaflet 776-3R, 2nd Edition, June 2009, Design requirements for rail-bridges based on interaction phenomena between train, track and bridge


779-1R UIC Leaflet 779-1R, 1st Edition, January 1996, Effect of the slipstream of passing trains on structures adjacent to the track

BD 49/01 Highways Agency. Design rules for aerodynamic effects on bridges


BS EN 1991-1-7:2006 Eurocode 1. Actions on structures General actions


NA to BS EN 1991-1-7:2006 UK National Annex to Eurocode 1. Actions on structures Accidental actions

INF TSI Infrastructure Technical Specification for Interoperability [currently under development]

ORR Guidance ORR guidance on the application of the common safety method (CSM) on risk evaluation and assessment – December 2012

RGD-2013-06 Policy statement on the relationship between the CSM for Risk Evaluation and Assessment and other risk assessment requirements – December 2013

ROGS The Railways and Other Guided Transport Systems (Safety) Regulations 2006 (as amended)

T750 RSSB research project Review of Euronorm design requirements for trackside and overhead structures subjected to transient aerodynamic loads

T988 RSSB research report Railway Bridge Design Requirements for GB Traffic

Utilities Contracts Regulations 2006

Utilities Contracts (Scotland) Regulations 2006