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Bridge Design to Eurocodes:
UK Implementation

Edited by **Steve Denton**

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Preface

In April 2010, the Eurocodes became the principal Standards for bridge design in UK. Over the previous two decades, a huge amount of work had been done to prepare for their implementation. In support of the transition to Eurocodes we set ourselves the objective of bringing important background and explanatory information into the public domain, and for this purpose we invited the leading technical experts in the UK to develop and present papers at a two day event held at the Institution of Civil Engineers (ICE) in London in November 2010.

This landmark conference provided the definitive background to the implementation of the Eurocodes for bridge design in UK, enabling engineers to understand key technical decisions taken in the development of the National Annexes and important sources of complementary information. In addition, papers were presented from major client organisations explaining their implementation strategies. Technical background was provided on companion European Execution Standards and illustrative examples were presented.

On behalf of the technical and organising committees, I would like to extend my thanks to all the authors who have contributed to these high quality proceedings. It was a privilege to have been able to draw together such an eminent and influential group at the forefront of Eurocodes development and implementation in UK. I hope that you find these proceedings a valuable resource and that they fulfil our ambition to serve as the essential and enduring record of the UK's implementation of Eurocodes for bridge design.

Dr Steve Denton

Chair, Technical and Organising committees, Bridge Design to Eurocodes – UK Implementation, ICE, London, November 2010
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UK IMPLEMENTATION OF EUROCODES FOR BRIDGE DESIGN – AN OVERVIEW

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Abstract

This paper provides an overview of the UK implementation of Eurocodes for bridge design. Some background to the Eurocode programme is provided and a brief overview of the structure of the Eurocodes is given. The steps that were necessary to support implementation in the UK are outlined, including an explanation of relationship between the Eurocodes and companion European Standards and other supporting national information and guidance.

Introduction

The implementation of Eurocodes has been recognised by the Institution of Structural Engineers[1] as, "the biggest change in codified structural design ever experienced in the UK". With all conflicting British Standards having been withdrawn by BSi at the end of March 2010, the Eurocodes have now become the principal codes for structural design in the UK. Over the past two decades, a huge amount of work has been done to prepare for their implementation in UK, a brief overview of which is provided. The focus here is the work undertaken to support the implementation of the Eurocodes for bridge design.

The design of bridges is expected to be at the forefront in the transition to the Eurocodes because of the requirements of the Public Procurement Directive (PPD), and in particular EU Directives 2004/18/EC, which covers contracts for services awarded by central government, local authorities and other public sector bodies, and EU Directive 2004/17/EC which covers works contracts awarded by entities operating in the water, energy, transport and postal service sectors. The Public Contracts Regulations 2006 implemented these EU Directives into UK law.

For projects bound by these PPD requirements, as most bridge design project in UK will be, a hierarchy is given for formulating technical specifications. National standards transposing European standards are the highest preference. The Eurocodes fall into this category, and major client organisations are therefore now specifying the use of Eurocodes for all newly procured bridge design projects[2],[3],[4].

The overall framework of national activities to support the implementation of the Eurocodes for bridge design is shown in Figure 1. It comprises: the development of the complete suite of European Standards for construction including the Eurocodes and their National Annexes, Product Standards and Execution Standards; the production and publication of Non-contradictory complementary information; client implementation strategies; and wider initiatives to support the profession. Each of these is discussed in this paper.

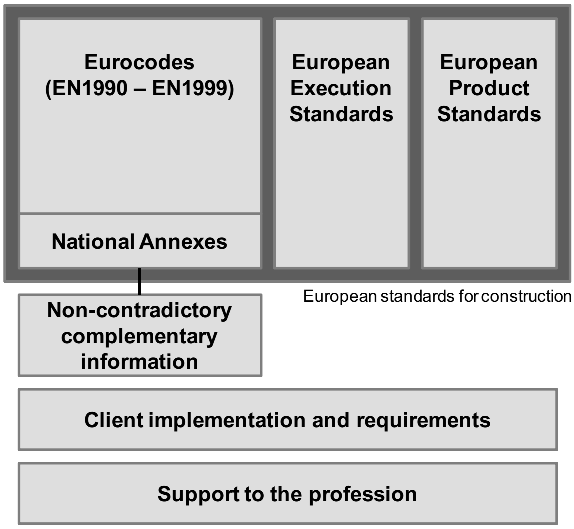

Figure 1: Framework of national activity for Eurocode implementation

Major contributions to the implementation of the Eurocodes in UK have been made by BSi committees and working groups, client organisations, industry bodies, engineering institutions, consultants, designers and academics. With much of the work unfunded, it has required key individuals to make significant personal contributions. It is not possible to acknowledge all these personal efforts, but in the field of bridges, the authors of the papers at this conference represent a core of the group of such individuals.

Background

The development of the Eurocodes began in 1975, as part of an initiative by the European Commission to eliminate technical obstacles to trade, and to harmonise technical specifications. In 1989, responsibility for the development of the codes was transferred to the European Committee for Standardisation (the Comite Europeen de Normalisation, or CEN), and in the early 1990's, the first generation of Eurocodes was published by CEN as European Pre-Standards (denoted ENV). The pre-standards had a limited period of validity, and were intended to co-exist with national standards.

In 2002, the first of the EN versions of the Eurocodes were made available by CEN for publication by the National Standards Bodies in each of the Member States (BSi in the UK). These superseded the ENV versions, and also initiated a process resulting in the withdrawal of conflicting national standards. Once the EN versions were released, there was a calibration period during which time each Member State set its own Nationally Determined Parameters

(NDPs), which were published in a National Annex, to accompany each part of the Eurocode as discussed below.

Publication of the National Annexes marked the beginning of a coexistence period, during which time both national standards and Eurocodes could be used. This period could last for up to three years, after which time the conflicting national standards had to be withdrawn.

The pace of implementation in the UK (and elsewhere in Europe) however meant that this transitional period was generally rather shorter. It is explicitly stated in the foreword to each of the Eurocode parts that all conflicting national standards should be withdrawn by March 2010. All conflicting British Standards were withdrawn at the end of March 2010.

BSi committees and working groups

BSi committees and working groups have played a central role in the implementation of the Eurocodes in UK. They have contributed to the drafting of the Eurocodes themselves, nominated UK representatives for CEN committees, determined the UK position in voting (see Denton[5]), drafted the National Annexes, and led the development of some key non-contradictory complementary information (NCCI).

The BSi committee structure that was in place for most of the period of Eurocode development and implementation is illustrated in Figure 2. Recently there have been some minor changes to this structure.

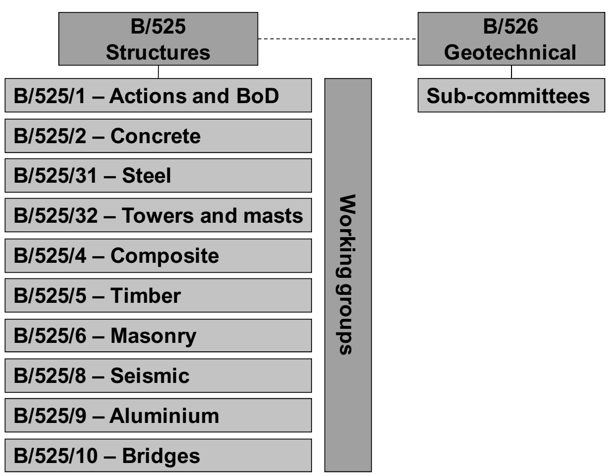

Figure 2: Framework of national activity for Eurocode implementation

Eurocodes

The Eurocodes comprise 10 Standards in 58 Parts for the design of buildings and civil engineering works. The 10 Standards are as follows and their relationship is illustrated in Figure 3.

- EN 1990 Eurocode: Basis of structural design
- EN 1991 Eurocode 1: Actions on structures
- EN 1992 Eurocode 2: Design of concrete structures
- EN 1993 Eurocode 3: Design of steel structures
- EN 1994 Eurocode 4: Design of composite structures
- EN 1995 Eurocode 5: Design of timber structures
- EN 1996 Eurocode 6: Design of masonry structures
- EN 1997 Eurocode 7: Geotechnical design
- EN 1998 Eurocode 8: Design for earthquake resistance
- EN 1999 Eurocode 9: Design of aluminium structures

Approximately 26 parts have content that can be relevant in bridge design. These are summarised in Table A.1 in Annex A.

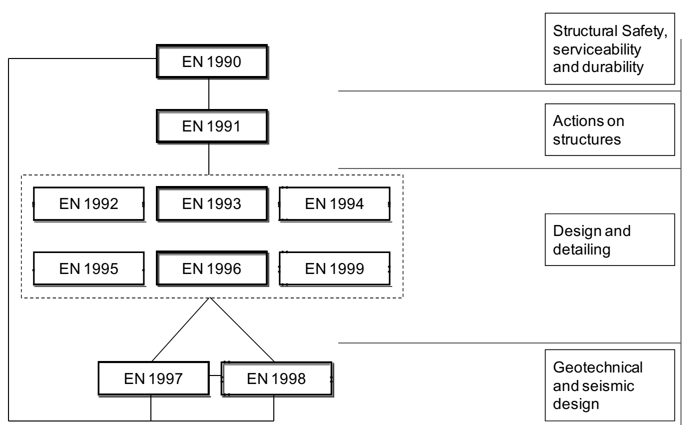

Figure 3: Overall structure of Eurocodes

Each Eurocode part was drafted by a Project Team, appointed by the relevant sub-committee of CEN TC/250 (see Denton[5]). These project teams were fairly small with typically around seven members. However, CEN members were invited to nominate National Technical Contacts (NTCs) to act as a focal point in reviewing drafts and providing comment back to the Project Teams. The substantial majority of Project Teams included a UK expert.

During their development, drafts of the Standards were reviewed by members of the relevant BSi committees and working groups and comments were submitted. In addition, prior to being finalised, all Eurocode parts were released by BSi as ‘Drafts for Public Comment’ (DPC).

Prior to publication, all CEN Standards require a positive vote from CEN members. The relevant BSi committee (or committees) determined the UK position on voting.

As a result of the large number of Eurocodes and the way material is distributed between different parts, navigation of the Standards is often a significant initial challenge. However, considerable efforts were made during drafting to achieve consistency between the structure, language and notation used in the different parts. For example, as illustrated in Figure 4a, all of the Eurocode parts are made up of similar elements, and as illustrated in Figure 4b, the first seven sections of the principal material-specific parts used in bridge design have common headings, enabling designers to readily identify the relevant section for the information they require.

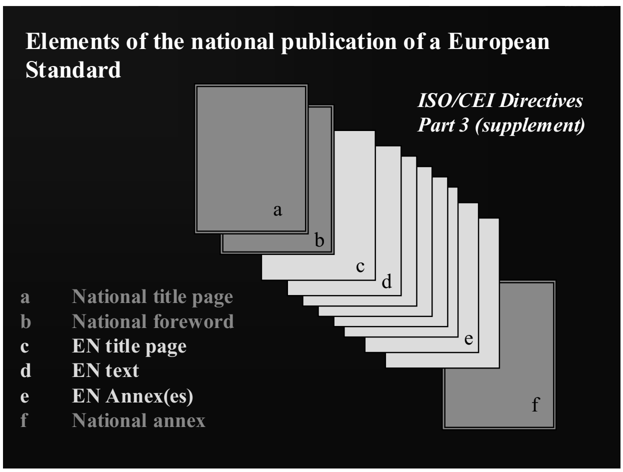

Figure 4a: Elements of each Eurocode part

	EN 1992-1-1 & EN 1992-2 **Concrete bridges**	**EN 1993-1-1 & EN 1993-2** **Steel bridges**	**EN 1994-2** **Composite bridges**
Section 1	General	General	General
Section 2	Basis of design	Basis of design	Basis of design
Section 3	Materials	Materials	Materials
Section 4	Durability and cover to reinforcement	Durability	Durability
Section 5	Structural analysis	Structural analysis	Structural analysis
Section 6	Ultimate limit states	Ultimate limit states	Ultimate limit states
Section 7	Serviceability limit states	Serviceability limit states	Serviceability limit states

Figure 4b: Common structure for first 7 sections of material-specific parts

National Annexes

The foreword to each of the Eurocodes states:

> 'Eurocodes recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.'

The Eurocodes provide a mechanism for Member States to exercise this right through the National Annexes.

The UK National Annexes were drafted by teams established by the relevant BSi committees or working groups, and then ratified by the BSi committee. Prior to publication, all National Annexes where made available as Drafts for Public Comment, and any comments received were reviewed and acted upon. The background to key decisions on the content of all the UK National Annexes relevant to bridge design are included in these conference proceedings.

The rules about what can be contained in a National Annex (NA) are strict and are set out in the foreword to each of the Eurocodes. National Annexes can only contain parameters that are left open for national choice, known as nationally determined parameters or NDPs. These NDPs are typically the values of parameters or country specific data, such as wind, snow or thermal maps.

There are a few cases, such as the 'Design Approach' used in the ultimate limit state verification of situations involving geotechnical actions or resistances (see Denton et al[6]), where the Eurocodes offer different procedures and the choice of approach is open to national determination. More usually, where the Eurocodes offer alternative approaches the decision on the appropriate choice will be open to the designer, possibly in conjunction with the client or a relevant authority.

It is easy to tell that a parameter is an NDP because a note below the relevant clause will state that the parameter is open for national choice and its value may be found in the National Annex, see for example the extract from BS EN 1992-1-1 below. In the majority of cases, a recommended value is also given. These recommended values have been included to assist the drafters of NAs and promote consistency across different member states. Member States have been encouraged to adopt recommended values, although the final decision rests with them.

In the example below, the parameter θ, which here refers to the angle of the truss used in verifying the shear resistance of a concrete member, is a nationally determined parameter and countries are allowed to specify bounds on acceptable range of values of θ. The recommended values correspond to a range from approximately 22^{o} to 45^{o}. A designer encountering this clause needs to look in the National Annex to find the national decision on the range of acceptable values of θ.

(2) The angle θ should be limited.

Note: The limiting values of $\cot\theta$ for use in a Country may be found in its National Annex. The recommended limits are given in Expression (6.7N).

$$1 \le \cot\theta \le 2{,}5 \qquad (6.7N)$$

Extract from BS EN 1992-1-1, Clause 6.2.3(2)

The foreword to each Eurocode part also explains that the National Annex may contain two further types of information. These are: decisions on the status of informative annexes; and, references to non-contradictory complementary information (NCCI) to assist the user to apply the Eurocodes.

The annexes contained in each Eurocode part, denoted 'e' in Figure 4a and not to be confused with the National Annex, may be either normative, meaning that they have the same status as the text in the main body of the Eurocode, or informative.

It is indicated immediately below an annex heading whether it is normative or informative. However, in contrast to the NDPs described above, for which it is quite clear that designers must look in the National Annex to find the national decision, there is no such note in informative annexes to explain that designers must consult the National Annex to find the national decision on its status. As a result, it appears that some designers are missing this important role of the NA, and assume that it is for them to decide whether they can use an informative annex. This is not necessarily the case.

Frequently, the NAs state that informative annexes may be used, leaving the choice on their application open to the designer. However, there are cases where NAs state that informative annexes should not be used and/or where alternatives are given.

The Eurocode foreword is clear that NAs should not themselves contain NCCI, but may include references to NCCI. The committees or working groups drafting the NAs in the UK have carefully considered which documents should be referenced as NCCI in the NAs. They include books, technical papers and guidance documents published by organisations such as the engineering institutions, CIRIA and other industry bodies. A particularly important series of NCCI documents for bridge designers have been published by BSi as 'Published Documents' or PDs. These are discussed below.

Queries have been expressed about the status of the National Annexes, often because in the versions published by BSi they are described as 'informative'. Recourse to the foreword of each Eurocode part removed any ambiguity. The forewords state that the National Annexes give the NDPs, '<u>to be used</u> for the design of buildings and civil engineering works <u>to be constructed</u> in the relevant country'.

In a Eurocode context, 'normative' strictly means that a requirement must be used in all Member States, and by definition NAs cannot therefore be normative. Following BSi's drafting rules, NAs are therefore described as 'informative'. However, from the foreword, it is clear that they are effectively 'nationally normative' (i.e. they must be used if a design is to be in accordance with the Eurocodes) and it is the NA relevant to the country where the project is built that must be used.

Non-contradictory complementary information

Non-contradictory complementary information or NCCI is the term used for the broad range of resources intended to support users to apply the Eurocodes. A huge amount of material has been published that can be considered as NCCI, and a good starting point for identifying it is the Eurocode Expert website (www.eurocodes.co.uk). NCCI includes books, technical papers and guidance documents published by BSi, engineering institutions and other industry bodies.

As described above, National Annexes can include references to NCCI, and the BSi committees responsible for NA development carefully considered the material to reference. The NCCI cited in the National Annexes should therefore be considered to have an elevated status and designers will be well advised to be aware of its content and make conscious decisions about whether or not to follow its recommendations and advice.

One particularly important series of NCCI documents for bridge designers have been published by BSi as 'published documents' or PDs. Some other National Standards Bodies produce similar types of documents, but the term PD seems unique to BSi and it is not particularly descriptive.

BSi Published Documents are supporting document generated by a BSi committee. They sit along side other BSi publications to provide greater insight and better comprehension of intent. They contains supplementary information, including guidance and recommendations but not have the status of a Standard. As such, they are an ideal vehicle for publishing NCCI and BSi committees have used them to provide background to National Annexes, to retain Eurocode-aligned residual material from withdrawn British Standards and to provide supplementary guidance and recommendations.

For bridge designers the PDs are particularly important as they contain a significant amount of material formally contained in the superseded BSI standards and the Design Manual for Roads and Bridges[7] (DMRB), maintained by the Highways Agency and Devolved Administration.

In preparing for the implementation of the Eurocodes the Highways Agency, with the support of Parsons Brinckerhoff and Atkins, reviewed all affected DMRB documents and developed plans for them to be updated to align with the Eurocodes. During this process it was recognised that the DMRB contained a lot of design information and guidance that was not client-specific but rather served as de-facto national requirements. It was considered that this material would be more sensibly housed in NCCI documents with a national status, and it has therefore now been included in the series of PDs published to accompany the Eurocodes relevant to bridge design.

Background to the development of the PDs relevant to bridge design is included in these conference proceedings.

Product and Execution Standards

In developing the rules contained in design standards it is necessary to define requirements for material, products, workmanship and quality control upon which these rules rely. Often these requirements are not included within the design standards themselves. Instead they are included in companion standards. Such an approach has been adopted in the development of the Eurocodes, with these companion standards referred to as product standards and execution standards.

Thus, as indicated in Figure 1, there are essentially three pillars of European standardisation for construction. The Eurocodes are design standards; the Execution Standards set out requirements during construction; and, Product Standards set out the requirements for material and products.

An overview of key product and execution standards for concrete, steel and timber bridges respectively, together with background to their implementation in UK and the development of companion guidance, is provided in these proceedings by Jackson et al[8], Ogle and Bowden[9] and Davies and Lawrence[10].

Client implementation and requirements

As illustrated in Figure 1, the work of clients in developing their implementation strategies and defining their requirements has formed another key component of the UK implementation of Eurocodes for bridge design.

Implementation by the Highways Agency, Local Authority and Network Rail are described respectively in these proceedings by Takano and Flanagan Palan[2], Clapham et al[3] and Whitmore and McLaughlin[4].

Support to the profession

The final component of the UK's implementation of Eurocodes illustrated in Figure 1 is the provision of support to the profession. Very significant efforts have been made in developing books, guidance documents, software tools, websites and training courses. Again, the Eurocode Expert website provides a valuable route to such information (www.eurocodes.co.uk).

In this conference, a series of practice papers has been presented to provide further support, providing design illustrations and specific guidance on aspects of bridge design to Eurocodes in UK.

Eight practice papers are included in these proceedings covering: key concepts of EN1990[11], combinations of actions[12], the design of concrete slab elements[13], early thermal cracking[14], a concrete bridge design illustration[15], buckling analysis for steel bridges[16], a composite bridge design example[17] and an illustration of bridge abutment design[18].

Maintenance and future developments

The maintenance and future development of the Eurocodes is discussed by Denton[19].

Conclusions

Over the past two decades, a huge amount of work has been done to prepare for the implementation of the Eurocodes for bridge design in UK. This has included contributions to the development of the Eurocodes themselves, the development of National Annexes and NCCI, product and executions standards, the implementation of client strategies and the definition of their requirements and a broad programme of activities to support the profession. A brief overview of these activities has been presented, often with reference to companion papers in these proceedings where greater detail is provided.

Acknowledgements

The authors wish to acknowledge the considerable efforts of all those who have contributed to the development and implementation of the Eurocode for bridge design in UK.

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Annex A – Eurocode parts relevant to bridge design

Table A.1 summarises the principal Eurocode parts with content that can be relevant in bridge design. In special cases, there may be material in other Eurocode parts that is useful.

Reference	Name
Eurocode Basis of structural design	
EN 1990	Basis of Structural Design
EN 1990/A1	Annex A2 : Application for Bridges
Eurocode 1 Actions on structures	
EN 1991-1-1	General actions. Densities, self-weight, imposed loads for buildings
EN 1991-1-3	General actions. Snow loads
EN 1991-1-4	General actions. Wind actions
EN 1991-1-5	General actions. Thermal actions
EN 1991-1-6	General actions. Actions during execution
EN 1991-1-7	General actions. Accidental actions
EN 1991-2	Traffic loads on bridges
Eurocode 2 Design of concrete structures	
EN 1992-1-1	General requirements
EN 1992-2	Concrete bridges - Design and detailing rules
Eurocode 3 Design of steel structures	
EN 1993-1-1	General requirements
EN 1993-1-5	Plated structural elements
EN 1993-1-7	General - Strength of planar plated structures loaded transversely
EN 1993-1-8	Design of joints
EN 1993-1-9	Fatigue strength
EN 1993-1-10	Material toughness and through thickness props
EN 1993-1-11	Design of structures with prefabricated tension components
EN 1993-1-12	General - Strength of planar plated structures loaded transversely
EN 1993-2	Steel bridges
EN 1993-5	Piling
Eurocode 4 Design of composite steel and concrete structures	
EN 1994-2	General rules and rules for bridges
Eurocode 5 Design of timber structures	
EN 1995-1-1	Common rules and rules for buildings
EN 1995-2	Bridges
Eurocode 7 Geotechnical Design	
EN 1997-1	General Rules
Eurocode 8 Design of structures for earthquake resistance	
EN 1998-1	General rules, seismic actions and rules for buildings
EN 1998-2	Bridges

Table A.1: Principal Eurocode parts relevant to bridge design

Bridge Design to Eurocodes: UK Implementation
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EUROCODE IMPLEMENTATION – THE EUROPEAN CONTEXT

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Abstract
This paper describes the European context for the development and implementations of the Eurocodes. Key requirements, procedures and guidance that have shaped Eurocode developments are outlined from the European Commission, the European Committee for Standardisation (CEN) and the CEN Technical Committee charged with the preparation of the Eurocodes (CEN/TC 250). Consequences of these requirements and of decisions taken early in the development of the Eurocodes on their style and implementation are discussed.

Introduction
The development of the Eurocodes has been strongly influenced by requirements, procedures and guidance published by the European Commission, the European Committee for Standardisation (CEN) and the CEN Technical Committee charged with their preparation (CEN/TC 250).

This paper provides an overview of important documents that have shaped the development of the Eurocodes, focussing on those aspects that have had a direct influence on the style of the Standards and their implementation.

The consequences of some key decisions on implementation in the UK are examined towards the end of the paper. A more general overview of the UK implementation of the Eurocodes for bridge design is given by Denton et al[1].

Background
The construction industry accounts for a very significant proportion of economic activity across Europe. It has been assessed as accounting for 7% of total employment and 28% of industrial employment[2], and was therefore a natural focus for the European Commission in their efforts to eliminate technical barriers to trade across the European Community.

In 1975, the Commission decided on an action programme based on Article 95 of the Treaty of Rome with the objective of harmonising technical specifications for construction works. The Commission, assisted by a steering committee composed of experts from EU Member States, oversaw the development of the Eurocodes programme, which led to the publication of a first generation of European codes in the 1980s.

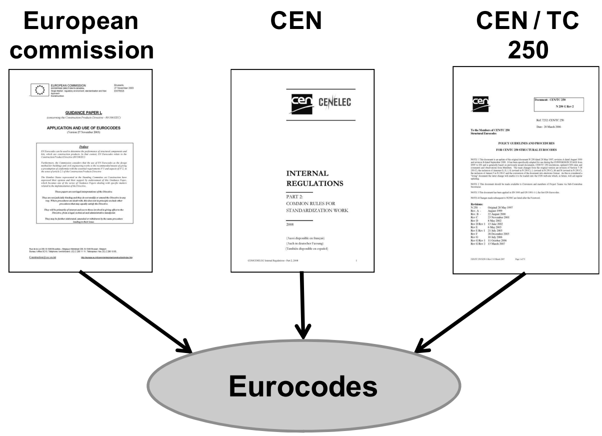

Figure 1: Key European influences on Eurocode development

After the adoption of the European Unique Act (1986), new European directives were established. These directives defined essential requirements, with responsibility for the development of Standards based on these requirements transferred to appropriate standardisation bodies. One of these directives, published in 1989, dealt with construction products.

In 1989, a special agreement between CEN and the European Commission transferred responsibility for the preparation and publication of the Eurocodes to CEN, thus providing them a future status of European EN standards. The subsequent development of the Eurocodes therefore followed CEN Regulations.

As explained by Denton et al[1], the Eurocodes comprise 10 Standards published in 58 parts. The Eurocodes include the facility to adapt their functioning to national settings and priorities through Nationally Determined Parameters (NDPs), given in a National Annex. In defining NDPs, Member States were expected to take account of justified differences in 'climate, geographic conditions (e.g. seismic risk), levels of safety, or traditions regarding the way of life prevailing in their territory'.

Following publication of the Eurocodes, the Public Procurement Directives (2004/17/EC and 2004/18/EC) required contracting authorities to allow their use for tenders falling within the remit of these Directives. Furthermore, it was intended that after the withdrawal of conflicting national standards (that was directed to occur by March 2010) the Eurocodes would become the only possible design codes for new publicly procured contracts.

However, in practice, some Member States were unable to meet the March 2010 deadline, often because of the time needed to modify their national regulations. At the present time, most Member States have implemented the Eurocodes, and the others are very well advanced in their preparations.

Framework for Eurocode development

CEN, European Committee for Standardisation

Overview

CEN (Comite Europeen de Normalisation) is the European standardisation body relevant to construction. Its National Members are the National Standards Bodies (NSBs) of the 27 European Union countries and Croatia plus three countries of the European Free Trade Association (Iceland, Norway and Switzerland). Thus, the National Member for the UK is BSi.

The standardisation system in Europe relies on the NSBs as the main focal point of access. The CEN Members have voting rights in the General Assembly and Administrative Board of CEN and they provide delegations to the Technical Board, which defines the work programme.

CEN National Members are bound by CEN Regulations. It is their responsibility to implement European Standards as National Standards. The NSBs distribute and sell the Standards and have to withdraw any conflicting National Standards within defined deadlines.

The CEN Technical Board (BT) is responsible for controlling the standardisation programme. It establishes the Technical Committees (TC) responsible for preparing CEN publications and approves their scope and programme of works. As discussed below, a key responsibility of the BT is to ensure coordination and avoid overlaps between Technical Committees. CEN currently has over 400 Technical Committees.

Internal regulations

The work of CEN is bound by its Regulations[3], which cover issues such as its organisation, committee structure, roles and responsibilities, internal and external relations, voting and language policy, and rules for the structure and drafting of European Standards.

The official languages of CEN are English, French and German. Technical committees can determine which of these languages is to be used in meetings and as the reference language for their documents. For the development of the Eurocodes, CEN/TC 250 adopted English as the reference language[4,5] and all meetings of TC 250 and its Sub-committees are held in English.

The development of CEN Standards follows a common process[3]. Eurocode drafts were made available for review and comment at various stages through their development, as described in CEN/TC 250 document N600[5]. At ‘Stage 32’ the first document was available, at ‘Stage 40’ the document was available for enquiry and at ‘Stage 49’ the document was available for vote.

At different stages the reference numbers of CEN drafts are prefixed to indicate their status. The prEN version is the version agreed by the Technical Committee that is forwarded to CEN National Members for public comment, known as CEN enquiry. The prefix FprEN is used for the approved final text following a positive formal vote by members.

Documents require a positive vote from CEN members before they are made available for publication. Although unanimity is sought in votes, a weighted voting procedure is used for European Standards and a positive vote requires 71% of weighted votes cast to be in favour (preferably of all members, but if necessary, only of EEA countries). The weighting of votes for each member country are given in the CEN Regulations[3].

A positive vote initiates a process leading to publications of an EN as a National Standard and the withdrawal of any conflicting National Standards within defined timescales.

CEN/TC 250 - Structural Eurocodes

Overview

As a CEN Technical Committee, TC 250 serves as a technical decision making body established by the BT to manage the preparation and ongoing maintenance of CEN deliverables in accordance with an agreed plan.

Technical Committees are composed of a chairperson, a secretary and CEN national delegations. National delegations are designated by the CEN members (i.e. NSBs), and comprise up to three delegates, one of whom acts as head of delegation. Other bodies may nominate observers to attend meetings, particularly where formal liaisons have been established.

Because of the extensive scope of TC 250's work, the committee has a reasonably complex structure as illustrated in Figure 2. Primary responsibility for each of the Eurocodes resides with a 'Sub-Committee' (denoted SC in the figure), which itself has a chairperson, secretary and national delegations. Drafting of the Eurocode parts was undertaken by 'Project Teams', typically with about seven expert members, reporting to the relevant Sub-Committee.

The one exception to this is EN 1990, which was considered so fundamental that responsibility for its development was retained by TC 250 with the drafting undertaken by a Project Team (PT EN 1990) reporting directly to it.

A smaller 'Co-ordination Group' oversees the work of the Sub-Committees to ensure alignment and address strategic issues. In addition, there are two active 'Horizontal Groups' with responsibilities for Eurocode parts relating to bridges and fire respectively. These Horizontal Groups work in a 'matrix' fashion with the Sub-Committees. During the drafting phase, there was also a Horizontal Group concerned with terminology, but this is now dormant.

The TC 250 Co-ordination Group is chaired by the Chair of TC 250 and its membership comprises the Chairs of all the Sub-Committees and Horizontal Groups, plus a small number of individuals fulfilling liaison roles. The Chairs of the Sub-committees and Horizontal Groups are also members of TC 250.

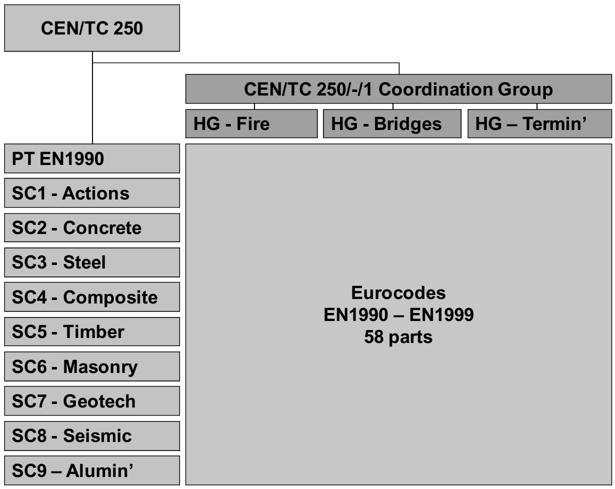

Figure 2: Structure of CEN/TC 250 during Eurocode development

CEN / TC 250: Document N250 Policy guidelines and procedures

The working procedures and policies of CEN/TC 250 are set out in an important internal committee document, referenced N250[4]. Parts of the initial version of this document were subsequently transferred to N600[5]. In addition to providing some background to the Eurocode programme, these documents provide specific guidance, aligned to the CEN regulations, on issues such as roles and responsibilities, programme, management and working approach of project teams, structure of the Eurocodes, drafting guidelines, committee procedures and Eurocode maintenance.

Document N250 (and subsequent N 600[5]) states that the working language of TC 250 is English, and that English is the reference language for the Eurocode parts. The Eurocodes were developed in English and subsequently translated into the other two languages of CEN (French and German). The fact that the Eurocodes were developed in English has had some significant consequences for UK implementation, as discussed later.

BT resolution BTS1 11/1992

Given the broad scope of responsibilities of TC 250 and the fact that other technical committees were developing Product Standards in parallel with the Eurocodes (See Denton et al[1]) , there was a clear risk of overlap and the potential for conflicting requirements. In line with its responsibilities, the CEN Technical Board passed an important resolution to address this risk, reference BTS1 11/1992.

An extract from this resolution is reproduced below, from which it is clear that TC 250 has overall responsibility for structural design rules and that any other technical committee wishing to incorporate design rules into their Standards should only do so with the agreement of TC 250. Unfortunately, this resolution has not always been honoured, but it does provide a mechanism to resolve issues when they arise and it serves as a clear statement of where the primary responsibility resides for structure design rules.

RESOLUTION BTS1 11/1992

Subject : Structural aspects - CEN/TC 250 "Structural Eurocodes" and other CEN/TCs

CEN/BTS1 decides that:

1. CEN/TC 250 has the overall responsibility for "structural design rules" in the building and civil engineering field.

2. If a CEN/TC (for products, execution, etc.) has a need to use structural design rules, it is asked to refer to the relevant EUROCODES whenever it is possible.
If it needs additional rules, the related CEN/TC is asked to define the work needed to be done in accordance with the agreed programme of work and to define the mode of cooperation with CEN/TC 250 (Annex I of Doc. BTS1 N 290).

Extract from CEN BT resolution BTS1 11/1992

European Commission - Guidance Paper L

Although the Eurocodes are Standards developed by CEN, their development was instigated by the European Commission and the Commission has retained a strong interest in their development. This is largely because of the close links that the Eurocodes have to various European Directives and Regulations. The Commission has provided direction to Member States and the authors of the Eurocodes and related Project Standards through the publication of Guidance Paper L[6] entitled, ‘Application and use of Eurocodes’.

It is worthwhile noting here the difference between Standards and Regulations (either National or European). Regulations are legal instruments. The legal status of Standards depends upon the regulatory framework within which they operate, which varies between Member States. For example, in the UK, the Building Regulations do not require particular Standards to be used, although the use of those referenced as Approved Documents[7] is effectively encouraged. In other European countries, Standards hold a very different legal status, with their use being a legal requirement.

The European Commission published Guidance Paper L to set out the common understanding of the Commission and Member States on important implementation issues, particularly in the context of the requirements of the Construction Products Directive - 89/106/EEC. It was intended for enforcement authorities, regulators, national standards bodies, technical specification writers, notified bodies and industry.

The objectives of the paper included: giving guidance on the implementation and use of the Eurocodes to maximise their benefits; providing a framework for the writers of the Eurocodes and of related Product Standards; allowing Member States to choose the level of safety, durability and economy applicable to construction works in their territory; and, providing Member States guidance needed to prepare public contracts, in line with the requirements of the Public Procurement Directive.

As an illustration, Guidance Paper L encourages Members States to determine their National Determined Parameters in National Annexes (see Denton et al[1]) in a manner that will reduce artificial barriers to trade. It encourages Member States to use recommended values or values within recommended ranges where these are given, and to co-operate to minimise the number of cases where recommended values or methods are not adopted.

Intended benefits

The European Commission have summarised the intended benefits and opportunities arising from the Eurocodes in Guidance Paper L[6] , as follows:

- to provide a common understanding regarding the design of structures between owners, operators and users, designers, contractors and manufacturers of construction products;

- to facilitate the exchange of construction services between European Member States;

- to facilitate the marketing and use of structural components and kits of parts in Member States;

- to be a common basis for research and development in the construction sector;

- to allow the preparation of common design aids and software;

- to increase the competitiveness of the European civil engineering firms, contractors, designers and product manufacturers in their world-wide activities.

For those actively involved in the implementation of Eurocodes, and particularly those working in international markets and research, it is clear that some of these benefits are already starting to be realised.

Implications for UK implementation

Some of the decisions taken early in the development of the Eurocodes and the requirements of CEN and the Commission have had inevitable consequences on the style, content and implementation of the Eurocodes. Three of them will be considered here.

1. The decision to develop the Eurocodes was essentially a political one

The decision to develop the Eurocodes was a political one, initiated in response to the Treaty of Rome over 30 years ago. Once their development was underway, and in particular, once the responsibility for their publication had been transferred to CEN, it was all but inevitable that conflicting National Standards would eventually have to be withdrawn and the Eurocodes would become the principal Standards in the UK for structural and geotechnical design.

Furthermore, because, under the relevant European treaties, issues of safety remain for national determination, it was necessarily for the Eurocode system to enable countries to set, for example, their own partial safety factors. The National Annexes provide the mechanism to meet this need.

In the later stages of development there has been disquiet expressed by some about the effects of the implementation of the Eurocodes on UK industry. In reality, by this time, the course was already set. For this reason, irrespective of the validity or otherwise of some of these concerns (and some of them have been rather ill-informed), the author and others working in the field have argued that it is most important for the UK to focus its efforts, at a personal, team, organisation and national level, on the successful implementation of the Eurocodes rather than being distracted by an essentially redundant debate about their merits.

2. The Eurocodes were developed in English

The reference language for the development of the Eurocodes was English. This had many advantages for the UK: it greatly assisting the involvement of experts from UK in their development; almost all of the Project Teams drafting the Eurocode parts had a UK member, so at least one had English as their first language; the English language versions are recognised as the reference versions in any case of dispute; and, the UK did not need to translate the Standards.

There was, however, a price to be paid. As the reference version, it was essential that the English language version of the Eurocodes could be translated into other languages without ambiguity. There are terms that have been used in past UK practice that, in reality, have been ambiguous, and it was necessary for these ambiguities to be avoided. This had resulted in some inevitable changes in terminology.

For example, in past UK practice the term 'load' has often been used to refer to both imposed loads (*i.e.* externally applied forces or self weight) and also imposed deformations (*e.g.* those arising from thermal expansion/contraction or support settlements). The Eurocodes address this ambiguity by using the term 'actions' as the generic term that covers both loads and imposed deformations. The term 'load' remains entirely valid in a Eurocode context, but is only used for loads.

3. The Eurocodes required a positive vote from CEN members

In accordance with CEN regulations, each Eurocode part required a positive vote from CEN members. The challenge of building consensus amongst CEN members is a very significant one, particularly considering the different engineering heritages, educational systems, predominant structural forms and ground conditions, and variations in customs and practice across Europe. Perhaps inevitably, the need to build consensus led to a degree of pragmatism in offering flexibility in allowing alternative methods (expressed as application rules) to be used to fulfil the principles of the Eurocodes. More significantly, it can be understood to have had a direct impact on the general style of the Eurocodes.

The authors of Standards have a decision to take at the outset of their development. As illustrated in Figure 3, Standards can vary in style from documents that are more procedural and provide 'step-by-step' or 'recipe-book' style requirements to those that focus solely on the principles that need to be satisfied. In reality, most Standards occupy a location towards the centre of the continuum between these extremes, although there still remains quite a variation in style around the world.

As suggested in Figure 3, on first use simple 'recipe-book' Standards are often favoured as they are easy to understand and easy to use, whereas principles focussed Standards can initially seem more complicated and cumbersome. In later use, 'recipe-book' Standards become less attractive because they can be overly prescriptive, have limited scope of application and are less efficient (as they typically use the same simple methods for a range of design situations). In contrast, once they are understood, principles focussed Standards become increasingly attractive because they tend to be more rational, consistent, flexible and efficient.

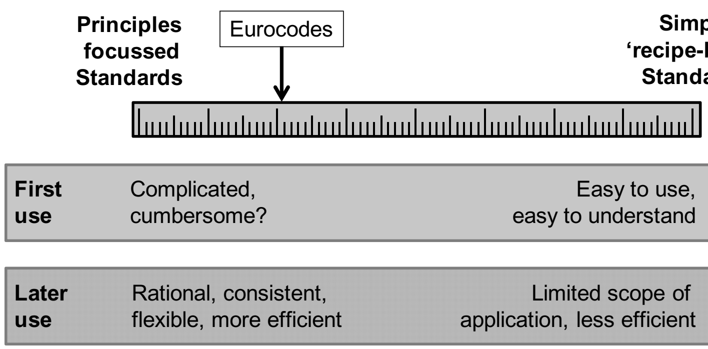

Figure 3: Style of Standards

It is far more straightforward to build international agreement on principles that it is to build consensus on methods to satisfy these principles. Thus, it was perhaps inevitable, given the CEN regulations on voting, that the Eurocodes would emphasise principles. Whilst application rules are given throughout the Eurocodes to satisfy the principles, their detail tends to be more limited in those areas of greater international variations in past practice and where it was more difficult to build consensus (such as in geotechnical design, see Denton et al[8]).

Maintenance and future developments

As noted previously, in accordance with CEN regulations, Technical Committees retain responsibilities for ongoing maintenance of their documents. Specifically, committees must undertake periodic reviews of all their Standards at least every 5 years, and remain formally responsible should questions of amendment and interpretation arise pending the next periodic review. Further detail concerning the maintenance and future development of the Eurocodes is provided by Denton[9].

Conclusions

An overview has been provided of key requirements of the European Commission, CEN and CEN/TC 250 that have shaped the development of the Eurocodes. The implications of aspects of these requirements on the style of the Eurocodes and their implementation in UK have been discussed.

Acknowledgements
The author would like to thanks his colleagues on CEN/TC 250 for many useful discussions that provided the basis for this paper.

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EUROCODES IMPLEMENTATION BY THE HIGHWAYS AGENCY

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Abstract
The paper describes the Eurocodes implementation process adopted by the Highways Agency (HA). It explains the objectives and principal aspects of the HA's strategy and key findings of some of the studies undertaken. The scope of application of the Eurocodes on Highways Agency schemes is also discussed.
The process that has been followed for updating the Design Manual for Road and Bridges (DMRB) is explained, together with the current status of relevant documents and plans for the future.

Introduction
Eurocodes are the suite of European Standards covering structural design of all civil engineering works, including bridges.

The UK, as a member of the European Union, is obliged to comply with the requirements of the Construction Products Directive (CPD) and Public Procurement Directive (PPD), which set out the status of European Standards in the member states.

The British Standards Institution (BSI) withdrew UK National Standards that were in conflict with Eurocodes on 31 March 2010, at which point Eurocodes became the published and maintained structural design standards in the UK.

Since 31 March 2010 the Highways Agency has expressed its requirements for the design of new and modification of existing highway structures (including geotechnical works) in terms of Eurocodes.

Background Information to the Implementation Process

Highways Agency objectives
The Agency has been preparing for Eurocodes for several years, and has developed a strategy for their introduction.

This strategy is being executed by the Highways Agency in coordination with the Overseeing Organisations of Scotland, Wales and Northern Ireland, and in liaison with interested industry bodies. There are regular meetings with the representatives of the Overseeing Organisations to discuss progress on Eurocodes and their implementation.

The strategy translates into a number of different areas of work, which can be categorised as follows:

National Annexes: Contribute to the calibration of Nationally Determined Parameters for bridges-related UK National Annexes through BSI working groups.

Non-Contradictory Complementary Information (NCCI): Draft documents that will become NCCI referenced from the relevant National Annexes. This information will supplement the Eurocode requirements for bridge design in the UK.

Highways Agency standards and specification: Review and redraft the Overseeing Organisations' requirements for the design and construction of structures in a number of selected documents within the Design Manual for Roads and Bridges (DMRB) and the Specification for Highway Works (SHW), to align with the principles of the Eurocodes.

Bridge design studies: Undertake studies using the Eurocodes to design bridges to understand the effect of introducing Eurocodes on design practices, how they will affect the physical characteristics of structures and the resources required for the design, as well as informing the development of guidance including the Design Manual for Roads and Bridges (DMRB).

Training and Seminars: Develop and deliver internal training for the Agency's Technical Approval staff, and contribute to industry meetings/conferences to raise awareness of the Agency's implementation strategy and programme among the current and potential suppliers within the construction industry.

Several members of Highways Agency specialist technical staff were involved in the development of the Eurocodes as National Technical Contacts (NTCs). Their role was to collate national comments and act as focal points for BSI/UK. Standards covered included EN1991 (Actions – Ron Ko), EN1992-2 (Concrete Bridges – Colin George), EN1993 (Steel – Sibdas Chakrabarti), EN1994-2 (Composite Bridges – Geoff Bowden) and EN1997 (Geotechnical – Alex Kidd).

Highways Agency objectives in the implementation of Eurocodes were underpinned by the need to comply with the requirements of the CPD and PPD.

The Eurocodes are intended "…to become the European recommended means for the structural design of works and parts thereof, to facilitate the exchange of construction services (construction works and related engineering services) and to improve the functioning of the internal market." [1]

To this effect, the overall HA objective was to try and provide 'equivalent' replacement for BS5400 and the parts of the Design Manual for Roads and Bridges (DMRB) and the Manual of Contract Documents for Highway Works (MCHW) affected, to enable designs to be undertaken to Eurocodes, whilst maintaining safety levels.

Supply chain and managing expectations

Supply chain expectations

The needs and expectations of the supply chain were perceived to include the publication of user guides, examples and background documents to ease the transition to Eurocode design.

The treatment of assessments and compatibility with strengthening design that follows was also flagged up as a topic requiring further guidance. An Interim Advice Note (IAN) on Eurocodes implementation was progressed with the intention of providing guidance to address some of these concerns.

Other concerns that needed to be investigated and addressed included the economics of design including the costs during the learning curve/transition, and the economics of the structure constructed. It was also important for the Highways Agency to assess the impact of Eurocodes on the design and construction process. To this end a number of pilot parallel design studies were commissioned.
The availability of implementation tools, such as standard analysis packages/spreadsheets was something that the Highways Agency felt would be best addressed by industry.

Concerns about the availability of background information to Eurocodes to enable Departures to be justified and the ability to specify and procure (Specification for Highway Works and contracts) were also expressed. The Highways Agency has tried to address this by working to amend the MCHW and by contributing to the publication of background material in BSI Published Documents (PDs).

There was also the expectation that with the introduction of Eurocodes and the open marketplace that would result, there would be the opportunities for pan-European design teams.

Design prior to Eurocodes implementation

Prior to 31 March 2010, UK National Standards such as BS5400, as implemented by the DMRB and MCHW continued to be used where designers were developing options and feasibility studies.

The Highways Agency allowed the use of Eurocodes for the design of new and modification of existing highway structures before the withdrawal of UK National Standards. However, a Departure from Standard was required to be submitted by designers for agreement by the Technical Approval Authority (TAA) until all relevant Eurocodes and UK National Annexes had been published to ensure that all necessary design and execution standards, technical guidance and information were used appropriately.

In addition, to discharge the Agency's responsibilities under CDM to vet the competence of designers, TAA required designs to be subject to the requirement of a Category 3 independent check, as defined in BD2 (DMRB 1.1.1).

Use of Eurocodes on Highways Agency contracts

The Highways Agency has been expressing its requirements for the design of new, and modification of existing, highway structures (including geotechnical works) in terms of Eurocodes for:

> All types of Major Projects contracts, involving highways structures design, made after 31 March 2010;

All new highway structures designs procured through MAC contracts, where detailed design commenced after 31 March 2010;

All new highway structures designs procured through DBFO contracts, where detailed design commenced after 31 March 2010 (provided the design standards were not frozen for the capital works);

All new highway structures designs procured through S278 agreements, where detailed design commenced after 31 March 2010;

All new Proprietary Products and highway structures, where detailed design commenced after 31 March 2010 (unless otherwise agreed by the TAA).

For the DBFO schemes, where the DBFO Co is not carrying the risk of standards change, the implementation of Eurocodes on new designs is not mandatory. Where the HA require Eurocodes to be implemented, this will be agreed on a case by case basis.

Eurocodes are not used for structural assessments which continue to be undertaken in accordance with the relevant extant DMRB standards and best practice.

Work on DMRB standards

In order to enable design organisations to design bridges and highway structures to similar criteria as current UK practice, it was necessary to amend the DMRB to suit the Eurocodes and National Annexes within a relatively short timeframe. Interim revisions to DMRB using Interim Advice Notes would also assist in the implementation of Eurocodes on HA schemes whilst evaluating their use and effect**.**

DMRB BDs and BAs affected by Eurocodes were identified – mainly within Volumes 1 and 2 – relating to design of highway bridges and structures. Also affected were standards for bridge furniture and products such as joints, bearings and parapets, lighting columns, CCTV masts etc.

The main areas of change required for Eurocode implementation were: references to BS design and product standards that would be withdrawn on 31 March 2010, cross references to other affected BDs/BAs, deletion of requirements arising from new/revised Eurocodes requirements, BDs/BAs to be deleted where corresponding BSs were to be withdrawn and additional non-contradictory complementary information (NCCI) for Eurocodes.

The Highways Agency has no plans at the moment to pursue the use of Eurocodes for assessments, as the current DMRB advice notes and standards are sufficiently comprehensive and versatile to cover the assessment of various highway structures.

The number of affected BDs/BAs was such that it was not a realistic proposition to have all the necessary amendments in place in time for the withdrawal of conflicting standards in March 2010. The details of the amendments were therefore incorporated into the document "Interim Requirements for the Use of Eurocodes for the Design of Highway Structures" contained in IAN124[2] (to be published imminently). This provides an interim measure to address these changes by giving guidance on which BDs/BAs to use fully with Eurocodes, which ones to use partially, and which ones not to use with Eurocodes.

The intention is to have the IAN in place during the early designs to Eurocodes, and to make use of any feedback to improve the document prior to its publication as a Standard (BD100).

The amendments to Technical Approval documents and templates for use with Eurocode design will be addressed in the revised version of BD2 "The Technical Approval of Highway Structures", which is expected to be published in early 2011.

Work on MCHW

In order to enable design organisations to construct bridges and highway structures to similar criteria as current UK practice, it was necessary to amend the MCHW to suit the Execution Standards within a relatively short timeframe. Draft revisions to MCHW would also assist in piloting the Eurocodes on HA schemes to evaluate their use and effect on real schemes**.**

MCHW documents affected by the Execution Standards were identified. The work to complete the amendments to the affected documents is nearing completion, but will not be finalised until after the publication of PD6705-2. *Recommendations on the execution of steel bridges to BS EN 1090-2*.

Once the amendments are finalised, an IAN will be issued to provide the necessary revisions to the relevant parts of the MCHW.

Pilot parallel design studies and cost implications

Technical aspects

In order to understand the potential impact that the introduction of Eurocodes would have, including technical challenges, potential risks to delivery programme and budget, the Highways Agency commissioned pilot parallel design studies.

The studies also attempted to validate the completeness of available standards and guidance documents, to assess their clarity and usability, and to identify ambiguities, gaps and errors.

In general, the studies found that the Eurocodes would make little difference to common forms of bridges and highway structures in terms of member sizes. Compared on a like for like basis, the Eurocodes generally resulted in sectional resistances that were within 10% of the results from the British Standards. [3]

The outcomes from the studies also highlighted some common areas of UK practice that are not covered by Eurocodes. This informed the work being done to develop additional complementary guidance. Of particular note, for example, is that the Eurocodes do not contain comprehensive provisions on the distribution of wheel loads through fill over buried structures, surcharge effects due to traffic loading adjacent to structures, or for the design of integral bridges. Guidance on all of these aspects will be included in PD6694-1. *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004*.

Cost implications

The pilot parallel design studies also considered comparisons of resource, programme requirements and cost between designing to Eurocodes and to BS5400.
Whilst the overall cost increase on programmes and projects of work is expected to be less than 0.5% of works value, the cost and time impacts on individual projects may be more significant. These impacts have been evaluated over a notional four year period, and could vary significantly, depending on the Agency's programmes of work, project types and the preparation and experience of designers.

In the first year following the introduction of Eurocodes, early designs undertaken by less-experienced consultants (without a structured in-house Eurocodes training programme) could cost up to an additional 45% for one-off structures designs, and 35% for multi-structure projects or MACs. Designs undertaken by experienced consultants (with significant in-house Eurocodes experience and a structured training programme) could be expected to cost up to an additional 30% for one-off structures scheme, and 25% for multi-structure schemes or MACs.

The percentage cost increases indicated above relate to the costs of design process including design time, drawing preparation and Category 1 & 2 Checking.

Programme timing impacts are expected to be proportionate to these cost impacts. There will also be cost and time impacts arising from the need to apply more independent checking of design. These costs can, however, only be determined on a project or scheme specific basis.

No significant construction cost savings are expected from the introduction of Eurocodes within this four year period. The potential does however exist for modest construction costs savings, once designers become more experienced with Eurocodes and the larger supply chain that Eurocodes should facilitate.

Eurocodes training

The Highways Agency appointed Parsons Brinckerhoff to provide tailored training to the Agency's technical staff (Technical Approval staff, and Structures Policy staff). The HA did not provide training for its supply chain.

Eurocodes Implementation

Interim Advice Notes IAN 123/10 and IAN 124

Interim Advice Note 123/10[4] was issued in March 2010 to implement Eurocodes on Highways Agency schemes. This IAN provides contractual and procedural guidance on the use of Eurocodes for the design of new, and modification of existing, highway structures (including geotechnical works) on the English Strategic Road network.

IAN 124 will provide technical guidance and client requirements on the implementation of Eurocodes and includes the "Interim Requirements for the Use of Eurocodes for the Design of Highway Structures".

The document includes clarification of specific Eurocodes requirements, a list of clauses in Published Documents that constitute the default means of compliance, status of DMRB documents for use with Eurocodes and project specific information that need to be recorded.

Changes to technical approval

The Highways Agency has revised its requirements for Technical Approval of Highway Structures to ensure that the requirements align with the technical and cultural changes brought about by the use of Eurocodes for structural design, as well as to address feedback received on the use of BD 2/05 Technical Approval of Highway Structures.

These new TA changes will be introduced in the forthcoming amendment to BD 2.

Interim Advice Notes and Technical Standards and Regulations Directive 98/34/EC[5] (TSRD)

TSRD arises from agreement of the member states to consult each other before adoption of new regulations and is considered an instrument of transparency that enables elimination of barriers to trade. (EC Treaty Articles 43 and 49 require elimination of all measures that prohibit, impede or render less attractive freedom of establishment and freedom to provide services.)

The key points of this Directive are that it defines technical regulations and *de facto* technical regulations, which have to be notified. Once notified, there is a standstill period of three months prior to publication, to give member states or the Commission the opportunity to comment or give an opinion. Further standstill and notification periods may apply if comments or opinions are made.

Originally, the Agency planned for a single IAN to be issued to cover contractual, procedural and technical guidance. However, the publication of the IAN for the implementation of Eurocodes was held up in the Highways Agency's discussions with the European Commission (EC), and an internal reappraisal of the status of some of our documents with regard to the TSRD.

At that time, the European Commission (EC) had advised the HA that two of its previously issued IANs should have been notified under TSRD, whilst the HA had considered its documents as "specifications" applied under contract, and therefore outside scope of TSRD.

According to legal advice, which the HA sought out, many HA standards are *de facto* technical regulations. The result of the discussion and reappraisal was that more of our new standards and specifications would have to be notified to the EC than hitherto considered necessary. Non compliance would risk infraction proceedings.
Notified documents have to be submitted with their supporting reference documents and the risk exists that some reference documents may give rise to infraction issues or further standstill periods.

The Interim Advice Note and standards that were originally prepared for Eurocodes fell within this need to notify. Therefore a staged approach to the issue of information for the implementation of Eurocodes was adopted.

The first stage was the publication of IAN 123/10, which addressed the immediate contractual and procedural implications of Eurocode implementation. IAN 123/10 was carefully worded to ensure that it was not a *de facto* technical regulation and therefore outside the scope of TSRD. This meant that it could be published immediately without a need for notification.

The second stage is the publication of IAN 124, which addresses technical guidance and client requirements for Eurocode implementation. This is currently going through the notification/standstill process.

Dealing With Queries

The implementation of Eurocodes led to the Highways Agency receiving many queries regarding the contractual, procedural and technical aspects of Eurocodes.

The Highways Agency has set up a process for dealing with these queries. The most popular queries were disseminated via the Frequently Asked Questions section of the HA website.

Ongoing Work

The Agency's strategy for the development of the DMRB was to use the implementation of Eurocodes as an opportunity to consolidate and simplify the bridge design parts. Whilst the Highways Agency had previously been in a position to provide, through the DMRB, guidance on British Standards, including amending or superseding BS clauses, this will be impermissible under the Eurocodes. Instead, the DMRB can only contain information that is complementary to the Eurocodes, along with the Overseeing Organisations' additional (non-contradictory) requirements.

The review of the existing DMRB with regard to Eurocodes is ongoing and the Agency is taking the opportunity to rationalise the content with a view to making it consistent with the principles of Eurocodes. There will be fewer, more focused DMRB parts that contain the additional (non-contradictory) requirements of the Overseeing Organisations. Currently there are about 60 BDs and BAs related to the design of bridges and other highway structures. These are being scrutinised and categorised: some information relates solely to the British Standards and will therefore be superseded; some information contains the Overseeing Organisations' additional requirements and will be retained in a form complementary to the Eurocodes; some information is out of date and will be withdrawn; and some is useful guidance material, which will be retained in a complementary format.

Whilst the DMRB will contain less guidance, it does not necessarily mean that there will be less information available; rather, there will be different homes for design information. For example, some material previously contained in DMRB documents has already been published as PDs.

The ultimate objective is for the number of documents in the DMRB to be reduced and guidance will be contained in a few well focused documents targeted towards the essential requirements of the Overseeing Organisations.

DMRB documents will still be needed for assessments, until such time as the assessment Eurocodes are published. In the meantime amendments to the assessment standards to make them more compatible with Eurocodes may be undertaken, particularly with respect to modification of structures.

Looking Ahead

IAN 124 will remain in place for some time. This will give opportunity for feedback and improvements to the standard before it is published as BD100.

The revisions aligning the MCHW to execution standards will be published some time after the publication of PD6705-2. *Recommendations on the execution of steel bridges to BS EN 1090-2*.

The ongoing maintenance of the DMRB will incorporate revisions to align documents to make them compatible with Eurocodes.

For the foreseeable future, assessments of existing structures will continue to be undertaken using the standards in the DMRB, making use of versions of the standards (such as BS5400) that were current at the time of withdrawal. Further work by CEN (European Committee for Standardisation) on the development of Eurocodes is likely to include assessment of existing structures, glass and Fibre-Reinforced Polymers (FRP).

The Highways Agency will continue to be engaged in these developments and will continue to support the transfer of knowledge and promotion of best practice.

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LOCAL AUTHORITY IMPLEMENTATION OF EUROCODES

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Abstract
This paper describes guidance related to Eurocode implementation for Local Authority Bridge Engineers that has been produced by the Association of Directors of Environment, Economy, Planning and Transport (ADEPT) National Bridges Group[1]. The guidance covers the reasons why Local Authorities will be using Eurocodes, and sets out recommended approaches for specific challenges for Local Authorities. This paper gives an overview of the guidance and highlights specific areas applicable to Local Authorities such as Technical Approval.

Introduction
Almost all journeys in the UK start and finish on local roads, and highway structures on these roads are a key element of the UK transport infrastructure. ADEPT members represent Local Authorities who are responsible for the development and management of the local road network and the associated highway structure assets. The implementation of Eurocodes will bring significant changes to how these structures are designed and specified. This in turn raises the need for changes in processes for the procurement, technical control and management of the design and construction of structures.

ADEPT members, as structures owners, will be assessing the changes required to their policies and procedures due to the implementation of Eurocodes. A new guidance document has recently been produced by ADEPT National Bridges Group [1] to support a common understanding among ADEPT members, a consistent approach to the implementation of Eurocodes and management of their impact on the procurement, design and construction of structures.

This guidance document is intended to provide information regarding the important issues arising from Eurocode implementation, set out recommended approaches and provide assistance to successfully manage the transition to adopting Eurocodes for structural design. It describes potential impacts of Eurocode implementation on Local Authority organisations, processes and staff training needs. It also includes a model register of risks associated with Eurocode implementation.

The objectives of the Eurocode implementation for Local Authorities are as follows:
- Comply with relevant EU legislation
- Manage technical, commercial and programme implications
- Develop staff and support them during transition
- Mitigate risks and maximise benefits associated with Eurocode implementation

This paper provides an overview of some of the key recommendations for Local Authorities related to Eurocode implementation, including guidance on Technical Approval. Further more detailed recommendations are available from the ADEPT guidance[1].

Legislative Requirements

The implementation of Eurocodes is linked to two important pieces of EU legislation, which are intended to eliminate barriers to trade, improve the competitiveness of the European construction sector, and create an open market for construction products – the Construction Products Directive (CPD)[2], and the Public Procurement Directive (PPD)[3,4].

The CPD is intended to remove barriers to trade, in the form of national technical specifications for construction products, and is implemented in the UK by the Construction Products Regulations[5] 1991 (Statutory Instrument 1991 No. 1620). The Eurocodes relate to the CPD in two ways: firstly, as a means of satisfying the Essential Requirements of the Directive, particularly in relation to mechanical resistance and stability (ER1) and safety in case of fire (ER2); and secondly, as a framework for drawing up harmonised technical specifications for construction products.

The relevant directives referred to as the PPD in this paper are EU Directive 2004/18/EC[3], which covers contracts for services awarded by central government, local authorities and other public sector bodies, and EU Directive 2004/17/EC[4], which covers works contracts awarded by entities operating in the water, energy, transport and postal service sectors. The Public Contracts Regulations 2006[6] (the "Regulations") implemented these EU Directives into UK law.

Article 23-3(a) of Directive 2004/18/EC includes the following text (and similar clauses exist in EU Directive 2004/17/EC), which effectively requires the use of Eurocodes where possible:

Without prejudice to mandatory national technical rules, to the extent that they are compatible with Community law, the technical specifications shall be formulated:
(a) either by reference to technical specifications defined in Annex VI and, in order of preference, to national standards transposing European standards, European technical approvals, common technical specifications, international standards, other technical reference systems established by the European standardisation bodies or - when these do not exist - to national standards, national technical approvals or national technical specifications relating to the design, calculation and execution of the works and use of the products...

Technically, there are thresholds for contract values for the PPD to apply. The Public Contracts Regulations 2006[6] provide the applicable thresholds, which it is anticipated will be amended every 2 years. The thresholds applicable from 1st January 2010 for entities listed in Schedule 1 of the Regulations are £101,323 for Supplies and Services (as defined) and £3,927,260 for Works. For other public sector contracting authorities (e.g. local authorities) the thresholds are £156,442 for Supplies and Services and £3,927,260 for Works. The thresholds for procurement however do not apply to the CPD, which remains the default means of demonstrating adequate strength, stability, fire resistance and durability. Further information on both directives is available from the website of the Office of Government Commerce[7].

Since the end of March 2010, all of the National Annexes and other supporting documentation required for the implementation of the Eurocodes in the UK have been available, and the process of withdrawing the conflicting documents has begun. The Eurocodes are now the only fully supported set of standards available in the UK for the design of bridges and other structures. At the same time, public authorities across the EU are obliged to specify the use of Eurocodes for design.
It is therefore recommended that Local Authorities should require the use of Eurocodes for all newly procured structural design work.

Existing Structures and Aspects not Covered

While there are legislative drivers for the use of Eurocodes for new design, their use is not mandatory for works outside the scope of the documents.

The treatment of existing structures for assessment, strengthening, rehabilitation and modification is mentioned in BS EN 1990: 2002, **1.1(4)** see Figure 1.

> (3) EN 1990 is applicable for the design of structures where other materials or other actions outside the scope of EN 1991 to EN 1999 are involved.
>
> (4) EN 1990 is applicable for the structural appraisal of existing construction, in developing the design of repairs and alterations or in assessing changes of use.
>
> NOTE Additional or amended provisions might be necessary where appropriate.

Figure 1. Extract from BS EN 1990: 2002, 1.1(4)

Although this suggests that BS EN 1990: 2002 does cover these issues, there is typically insufficient coverage in the other Eurocodes and, in effect, the note under BS EN 1990: 2002, **1.1 (4)** concedes that the use of non-Eurocode requirements will often be appropriate for the treatment of existing structures.

At the moment the Eurocodes do not adequately cover the assessment of existing bridges and other structures, and as the UK has a comprehensive set of mature standards available for assessment (e.g. BD 21/01 for highway bridges[8]), bridge owners in the UK will be able to continue to use these assessment standards for some time to come. The long term aim is to develop Eurocodes for structural assessment; CEN/TC 250 Working Group 2 is tentatively anticipating that they could be available in 2015, although this may be rather an ambitious target.

Design for the modification of existing structures should use Eurocodes as far as possible, but it may still be necessary to use additional verifications based on assessment standards where there are any doubts regarding the adequacy of the existing construction. Eurocodes can be used to determine resistances in many areas, but it is necessary to understand the limitations of applicability which often relate to reliance on modern materials, detailing and tolerances, and workmanship standards.

Particular difficulties may arise for modifications such as deck widening projects. Previously, it was possible, through the use of Departures from Standards, to retain or replicate details of structures that did not conform to design standards, as set out in BD 95/07: *'Treatment of Existing Structures on Highway Widening Schemes'*[9]. This standard has not been written from a Eurocode perspective, so with the introduction of the Eurocodes, some of the processes described in BD 95/07 will be affected, and issues previously dealt with as 'Departures from Standards' may have to be considered as 'Aspects not covered'. However, most of the general principles of BD 95/07 will still apply.

In general, wherever there is an aspect outside of the scope of Eurocodes, whether related to the treatment of existing construction or materials or actions that are not covered by Eurocodes, then the approach will need to be defined in the Approval in Principle as an "Aspect not covered". Particular care will need to be taken to ensure that the proposed approach is compatible with the requirements of EN1990 for the basis of design, including the approach to combinations of actions, design situations and limit states as set out in that document.

Technical Approval

The evolution of Technical Approval procedures in the light of Eurocode implementation is a major concern to local authorities. In many authorities, structural design work relating to highway bridges has been out-sourced to private sector consultants, with the authority retaining the role of Technical Approval Authority (TAA), as part of their 'client' responsibilities. In such cases, the additional demands placed on the TAA to identify and understand the significance of the design options and choices proposed by the designer will clearly heighten this concern.

For schemes where design work has already commenced using the existing DMRB/British Standards (so-called 'legacy schemes'), the expectation is that the design will be completed using those standards (in accordance with the original Approval in Principle). One of the consequences of this approach is that the Technical Approval of a new scheme will be the first 'live' experience that many local authority bridge engineers/managers will have of the Eurocodes – another reason why proper preparation and guidance is so important.

The majority of local authorities have traditionally followed the guidance on Technical Approval provided in DMRB Standard BD 2/05[10]. The structure of this document and the layout of the model Approval in Principle (AIP), TAS schedules and Design and Check Certificates have remained broadly unchanged for many years, and are therefore familiar to many. The Highways Agency is currently leading a major revision of BD2, to bring it in line with the Eurocodes, but it is understood that the basic format of the AIP and other standard forms will be retained, where possible. While not being mandatory for use by local authorities, it is recommended that the forthcoming revision to BD2 should be used for Technical Approval to Eurocodes as appropriate. Further guidance on Technical Approval to Eurocodes is currently available in the ADEPT guidance document, including draft model AIPs for use with Eurocodes.

The principal amendments and additions to the AIP and other Technical Approval documentation result from changes in the design philosophy and terminology used in the Eurocodes, and in the documents referenced within the TAS. While the focus of Technical

Approval remains on the control of technical risk, particularly in relation to the assessment of safety and durability, opportunities have also been taken to simplify the process, where appropriate. The new form of AIP also provides a framework for recording the design options and choices to be used in the detailed design and check.

Changes to the AIP include:

- Description of structure – to include design working life;
- Inclusion of Consequence Class, Reliability Class and Inspection Level;
- Design Criteria – Loads will be replaced by Actions, including Accidental Actions and Actions during Execution;
- Structural Analysis – including methods of analysis for different design situations and limit states, and proposed range of soil parameters for earth-retaining elements;
- Checking – proposed Category of Check, and Design Supervision Level (as defined in EN1990).

There are fundamental changes to the content of the Technical Approval Schedule (TAS). A model TAS is included in the ADEPT guidance document. The documents that will now be referenced include:

- Eurocodes (including reference to specific Parts)
- Corresponding UK National Annexes
- BSi Published Documents (PD's)
- Relevant Product and Execution Standards
- Non-conflicting British Standards
- Non-conflicting DMRB Standards and Advice Notes
- Interim Advice Notes

The changes shown highlight the level of awareness of Eurocode principles required by the TAA, particularly in relation to design choices and options, in order to properly assess the suitability of the proposals.

Processes Affected by Eurocode Implementation

Each local authority will have its own set of internal processes governing the management of its business, some of which will relate to the planning, procurement and delivery of engineering works, where the use of structural design standards forms an essential part. To ensure successful implementation of the Eurocodes, it is important for each authority to understand how these processes will be affected, so that their implementation strategy can incorporate not only the 'technical' changes to the design process discussed in the previous sections, but also the procedural changes required to align with the new design codes, and associated European legislation.

Procurement and delivery of structures schemes

The considerations given to the project at each stage of the process relate to a number of areas, all of which may be affected by the implementation of Eurocodes to some degree. Some of the impacts are listed in Table 1.

Strategic objectives and scope	When defining strategic objectives, the use of Eurocodes and the impacts on value for money and long-term benefits should be considered.
Governance and stakeholders	A clear strategy for Eurocode implementation will help ensure that governance arrangements are applied throughout the supply chain.
Funding	Funding will be a significant issue for Eurocode design schemes in the short-term, as it is expected that the design fee will increase as designers will take more time to undertake early Eurocodes designs.
Resources	The ADEPT guidance provides criteria to assess potential suppliers' capabilities to design with Eurocodes. These may be used as the basis for processes that will help Local Authorities select suppliers with the right expertise and qualities to undertake design to Eurocodes.
Procurement and commercial issues	Local Authority procurement strategies may need careful consideration in the short-term to protect against excessive costs in certain types of projects (e.g. time and expenses projects). Robust criteria for selecting the right suppliers should be used. This is an important consideration at the early stage of scheme development.
Legal and consents	The impact of Constructions Product Directive and Public Procurement Directive in the context of technical specification (i.e. use of Eurocodes) must be understood and considered, with any restrictions identified and recorded for all projects to avoid legal implications. The ADEPT guidance also gives advice intended to ensure that the required level of technical governance continues to be provided for Local Authority structures schemes.
Engineering and technical issues	Technical risks arising from the use of new structural design standards, including the cultural changes brought about, can be managed through the revised Technical Approval process. This will require information such as consequence classes, reliability classes, options and choices in design to be recorded. Increasing the amount of independent checking of designs in the interim period should also help mitigate the technical risks arising from the introduction of the new design standards.
Project and programme management	Although Local Authority processes for project/programme management are not expected to require changes, effective management of the issues identified in the ADEPT guidance will help them to continue to run their various projects and programmes.

Table 1: Impacts of processes for procurement and delivery

Structural design processes

The impact of the introduction of Eurocodes will clearly be most noticeable in processes directly related to structural design. While the engineering concepts and principles behind both the existing British Standards (as implemented by the DMRB) and the Eurocodes are fundamentally the same, the Eurocodes incorporate more up-to-date research findings, and certain key aspects of the design process will be different from current practice. In addition, Eurocodes adopt a less prescriptive approach, giving designers more flexibility in the choice of design options and analysis methods. While this should promote more innovative and economic design, this may place an additional onus on the Technical Approval Authority to consider (and understand) such choices within the technical approval process.

It is quite likely that the introduction of Eurocodes (and other related European legislation/standards) will also require changes in the wider procedures surrounding the design process, for example, within the authority's Quality Management System.

Specification and execution (construction)

Generally the Highways Agency's Manual of Contract Documents for Highway Works is used for the specification and construction (or "execution") of its highway structures. These implement existing British Standards for the specification for concrete, steel and geotechnical works. New European Execution Standards have been published, giving rules for specification issues including construction tolerances and execution classes used for designs to Eurocodes. The Specification for Highway Works is being reviewed and it is anticipated that amendments to align with new requirements in the Execution Standards will be incorporated in future revisions.

Management of structures

Since its launch in 2005, the Code of Practice for the Management of Highway Structures[11] has provided comprehensive guidance for bridge owners on all aspects of the management of highway bridges, and has been widely promoted for adoption by all local authority bridge owners in the UK. Complementary guidance has also been issued[12] to support the Code of Practice and provide updates on any new developments, including a brief section on the implementation of Eurocodes.

While many bridge management procedures (such as inspection, assessment, and structures asset management) will not be directly affected by the implementation of Eurocodes, activities such as bridge strengthening and rehabilitation may be affected (as discussed previously), and the European Execution Standards may have an impact on routine and remedial bridge maintenance activities. It is anticipated that these issues will be incorporated into future amendments to the Code of Practice.

Management of Design and Checking

In order to manage the design process effectively, and to minimise the risks associated with procuring Eurocode designs, the Local Authority Client and Technical Approval Authority should be assessing supply chain experience and capabilities. While Eurocode training programmes have been available for some time, experience of using Eurocodes for 'live' designs in the UK is still limited. Criteria are suggested in the ADEPT guidance for assessing the suitability of suppliers to deliver Eurocode designs.

In the short term it is recommended that a greater degree of independent checking should be carried out for Eurocode designs. Further advice is provided in the ADEPT guidance.

Staff involved in the procurement and management of Eurocode designs should appreciate the fundamental principles of Eurocode design, for example, philosophical changes such as design situations, new physical limit states and combinations of actions. These principles are explored in the ADEPT guidance and the paper by Denton and Gulvanessian[13].

Managing Cost and Programme

It is likely that, in the short term, the design of structures using Eurocodes (and the associated checking and approval procedures) will take longer than designs using the previous national standards, because of unfamiliarity with the new codes. This will therefore lead to a corresponding increase in design fees. For projects due to be undertaken during the 'transition' phase, adequate allowance for this increased time and cost needs to be included in the programme and budget.

The rate at which the cost premium for using Eurocodes will fall will depend on the frequency with which the new codes are used, and the effectiveness of the Eurocodes training programme that the design organisation puts in place. For local authorities undertaking structural design using in-house teams, the timing and content of training courses, and those who attend them, should be chosen to ensure maximum efficiency of the transition process. For authorities who procure structural design services from external providers, the selection of consultants should include:

- An assessment of their strategy for implementing Eurocodes.
- Their current levels of knowledge and experience.

Where the consultant has already been engaged on a framework or term contract basis, the local authority client should regularly review the consultant's training programme and their progress in gaining knowledge and experience in the use of Eurocodes, to ensure that the service being delivered meets not only the required level of quality, but also represents good value for money.

Overall it is reasonable to expect that the introduction of Eurocodes, and the associated European Product and Execution Standards, could result in overall cost savings for clients in the long term. The additional flexibility contained within the Eurocode design philosophy can provide designers with more opportunities for innovation, and to be able to incorporate the latest developments in methods of analysis and materials technology into their designs. To realise these potential benefits, client authorities should encourage opportunities to embrace improved design processes as described in the Eurocodes.

Managing Design Undertaken by Third Parties

In addition to managing the design process for new structures arising from their own improvement programmes, local authorities frequently have to act as the Technical Approval Authority for highway structure design proposals prepared by external third parties. These are generally privately-funded infrastructure developments, which will subsequently be adopted as part of the local highway network, and so become owned and maintained by the local authority. These will typically be bridges or other structures to support access roads for residential housing estates, or commercial/industrial development sites. This is generally covered by Section 38 or Section 278 of the Highways Act[14].

The local authority will ultimately become responsible for these structures, so it is essential that close control is exercised over all stages of the project to ensure that the safety of those using the structures is not compromised, and also that they have been designed and constructed in a way which does not render the authority liable for significant maintenance costs in the future. A commuted sum, based on recent ADEPT guidance[15], is usually paid by the developer when the structure is adopted. This is to cover future maintenance costs, but the structure should still be designed and specified to minimise future costs over its design life – generally 120 years.

As such structures are not being procured at the public's expense, it could be argued that the use of Eurocodes may not be mandatory. However, since April 2010, the previous national standards for structural design are no longer supported, hence it is reasonable for the Local Authority to insist on the use of Eurocodes for all new structural design.

The TAA will need to give the developer clear guidance on what is required to enable approval for the proposals to be granted. The TAA should also take steps to check the level of competence and experience of the consultant in the use of Eurocodes.

Training and Staff Development

Effective training and development of staff is an essential requirement for the successful implementation of Eurocodes. This does not mean that everyone needs to be trained in the same way, or to the same level the specific requirements of each group of staff whose roles are affected by the Eurocodes should be identified, and training then tailored to meet those needs.

This may range from a short briefing session, to raise general awareness; to formal training courses in the detailed technical aspects of the new codes will be required. Other groups of staff may need specific training in the legal, contractual or commercial aspects of the codes, and associated European legislation. In all cases it is important that training covers the overall framework of the Eurocodes, and the reasons for their implementation, as well as the 'job-specific' elements.

The learning process will be more effective if those who receive the training have the opportunity to put it into practice within a short space of time. This may help identify the best timescale for delivering training. A generic review of training needs, identifying the particular requirements for different groups of staff, is provided in the ADEPT guidance[1].

Risks

The implementation of Eurocodes is accompanied by a number of risks for ADEPT members. For local authorities with in-house consultancy capability, these will include the technical risks associated with any change in design standards. For other authorities it may be the procedural and commercial risks that will be of greater significance. As client organisations and structures owners, it is important that these risks are recorded and managed, so that appropriate mitigating measures can be put in place.

A model risk register has been included in the ADEPT guidance, illustrating those risks which are likely to be common to most ADEPT members, including notes on the significance of each risk, who is best placed to own the risk, suggested mitigation measures, and the potential consequences of non-action. The risks are grouped into a number of categories, as summarised in Table 2.

General	Failure to achieve a consistent approach to Eurocode implementation within the authority, and among partner organisations
Legal and Contractual	Failure to comply with relevant EU legislation. Changes of standards used in on-going contracts and frameworks. Inconsistencies between existing standard terms and conditions, and the requirements of EU legislation and directives.
Financial	Potential increase in design costs due to lack of familiarity in the use of Eurocodes.
Programme	Delays to design phase of projects, due to lack of familiarity in the use of Eurocodes.
Technical/Design	Failure of designers to choose the appropriate design approach or analysis method Shortage of suitably trained senior engineers and design team leaders to effectively check and supervise the design process Lack of understanding amongst Technical Approval staff of Eurocode design principles and the differences from BS5400 design.
Quality	Lack of experience in the application of Eurocodes could lead to a reduction in quality of the design process, or in the design solution delivered Inappropriate selection of Eurocode options and choices by designers or Technical Approval staff, could result in design solutions which do not give good value for money, in whole life terms.
Guidance and Training	Failure to ensure that all staff who may be affected by the implementation of Eurocodes, are given sufficient training and guidance, appropriate to their specific role
Communication	Failure to effectively manage communications regarding Eurocode implementation
Reputation	Damage to the authority's reputation resulting from ineffective implementation of the Eurocodes, leading to poor delivery of service, or poor value for money.

Table 2: Risks associated with Eurocode Implementation

Conclusions

Local Authorities should use Eurocodes for the design of all projects procured since April 2010. The ADEPT guidance covering the implementation of Eurocodes[1] provides practical information and guidance about the changes required to implement Eurocodes. There are a number of issues of particular interest to Local Authority Engineers, including Technical Approval and the treatment of existing structures.

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RAILWAY IMPLEMENTATION OF EUROCODES

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Abstract

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 Eurocodes once the national standards are withdrawn. Directive 2004/17/EC is implemented in the UK through the Utilities Contracts Regulations 2006 and the Utilities Contracts (Scotland) Regulations 2006. These are the Regulations that apply to Network Rail as the Infrastructure Manager.

Now that BS EN 1990:2002 + A1:2005 and BS EN 1991-2:2003 are available and BS 5400-2:2006 has been withdrawn, Railway Group Standard GC/RT5112 ‘Rail Traffic Loading Requirements for the Design of Railway Structures’ will require railway bridges to be designed using the Eurocodes.

The paper will describe the arrangements and policies that have been put in place by Network Rail to implement the requirements of the Regulations and the Railway Group Standard to manage the design of railway bridges using the Eurocodes

Legal Framework

The application of the Eurocodes to Network Rail (NR) works stems from a number of pieces of legislation and railway regulation from four key sources, European legislation, UK national legislation, UK railway specific legislation and UK standards bodies. Figure 1 aims to set out a pictorial representation the legislation and requirements from each source.

European legislation

Public Procurement Directive

The requirement to adopt the Eurocodes for structural design in the UK originates from the publication of Directive 2004/17/EC of the European Parliament and of the Council of 31st March 2004 - Co-ordinating the procurement procedures of entities operating in the water, energy, transport and postal services sectors. This directive is referred to as the Public Procurement Directive[1] (PPD).

Article 34 of the Directive refers to Technical Specifications and requires the specifications to ‘… afford equal access for tenderer’s and not have the effect of creating unjustified obstacles to the opening up of public procurement to competition’.

The Article goes on to list a hierarchy for the application of standards to the specification. Where prescriptive standards are required, these should be in the order of ‘…national standards transposing European standards, European technical approvals, common technical specifications, international standards, other technical reference systems established by the European standardisation bodies or — when these do not exist — national standards, national technical approvals or national technical specifications relating to the design, calculation and execution of the works and use of the products'.

The Article makes clear that the contracting entity cannot reject a tender based on other specifications but that the supplier must be able to demonstrate equivalence and it is suggested that this may be done through the submission of a technical dossier supporting the case. However, in the case of design standards such as Eurocodes, demonstrating equivalence can only realistically be done with reference to specific designs rather than standards in general, and for this reason it is considered unlikely that it will be a worthwhile avenue for designers to pursue.

The application of the PPD to NR is through Annex IV which lists the contracting entities covered by the Directive, albeit the reference is to Railtrack. The work activity listed in Annex Xll covers bridges amongst others and the services listed in Annex XVII are engineering services.

Interoperability Directive

The Railways Interoperability Directive 2008/57/EC[2] (which replaces 96/48/EC[3] and 2001/16/EC[4]) makes provision for infrastructure (including bridges) on high speed and conventional railways. The way in which this is to be achieved, is through the use of the Technical Specifications for Interoperability[5] (TSIs). The TSIs are based on 'structural sub-systems' and in the case of railway bridges, the relevant 'structural sub-system' is infrastructure.

Directive 2008/57/EC is railway specific and therefore takes precedence over other EC directives impacting on railway infrastructure. In practice, this makes no difference because the design criteria and load models specified in the TSIs, refer to the relevant clauses in BS EN 1990 Annex A2[6] and BS EN 1991-2[7].

The Directive will apply to NR as the 'Infrastructure Manager' where the 'Competent Authority' (in this case the Department for Transport) has decided that railway infrastructure must conform with the TSIs. As such decisions are not taken retrospectively, the TSIs will only affect new or upgraded infrastructure where it has been decided that the infrastructure must conform with the TSIs.

Construction Products Directive

Application of Eurocodes to projects can also be used to demonstrate project compliance with the Construction Product Directive (CPD), Council Directive 89/106/EEC[8]. It should be noted that this Directive is not mandatory at present through UK legislation but that the EU Commission is in consultation with Member States on replacing the Directive with the Construction Product Regulations[9]. It is expected that the main provisions of the Regulation will apply in the UK from July 2011.

UK legislation

The PDD is implemented in the UK through the Utilities Contracts Regulations 2006[10] and the Utilities Contracts (Scotland) Regulations 2006[11]. Both of these regulations reiterate the requirements of the PDD in relation to technical specifications but it is worth noting the subtle changes in two areas. Firstly, Network Rail is now specifically cited in Schedule 1, Category 7, Part Q. Secondly, regulation 12(14), clarifies the need to include a technical dossier to support tenders using other specifications to prove the 'equivalence' thereof.

Railway specific legislation

The Railways Act 1993[12] was the enabling Act for railway privatisation in the UK. This Act, amongst other things, established the requirement for NR to hold a licence to operate. The Office of Rail Regulation grants the licence[13] which, in Part F, Condition 22.2 requires NR to be a member of the Railway Safety and Standards Board (RSSB) and to comply with Railway Group Standards.

The Railways and Other Guided Transport Systems (Safety) 2006[14], known as ROGS, requires those with responsibility for safety to maintain a Safety Management System[15] (SMS). ROGS also requires operators to demonstrate that they have procedures in place under the SMS to bring new or altered infrastructure safely into use. This requirement includes specific reference to relevant technical and operational standards, including national safety rules.

The Railways Interoperability Directive 2008/57/EC has been implemented in the UK through The Railways (Interoperability) Regulations 2006[16]. The regulations make provision for 'placing infrastructure sub-systems into service' through the Technical Specifications for Interoperability (TSIs) for infrastructure and, where relevant 'national technical rules'.

Standards

Railway Group Standards are managed by the RSSB which was formed in 2003 with the primary objective to '…lead and facilitate the railway industries work to achieve continuous improvement in health and safety performance of the railways in Great Britain….' [17] Railway Group Standards can be used to notify both National Safety Rules and National Technical Rules to the European Commission.

The Railway Group Standard GC/RT5112:2008 Rail Traffic Loading Requirements for the Design of Railway Structures[18] references the PDD and opens the ability to design to either BS5400-2:2006[19] or BS EN 1991-2:2003[20] plus BS EN 1990:2002(A1) [21] '…until such time as BS5400-2:2006 is withdrawn'.

British Standards Institution (BSi) is a member of the European Committee for Standardization[22] (CEN) and as such has a duty to support the Eurocodes. European standards produced by CEN must be implemented by the national standards bodies of all member countries. When published by the national body, BSi in the case of the UK, the European standard is given the status of a national standard and a national reference is added to the European standard number on the cover and on any other pages included in the national edition.

Following the publication of all the Eurocodes and supporting documents, BSi was obliged to withdraw conflicting standards[23], which it did on 31st March 2010. Although withdrawn documents will continue to be made available and remain in the BSi catalogue for historical information purposes BSi are no longer supporting or maintaining the withdrawn standards.

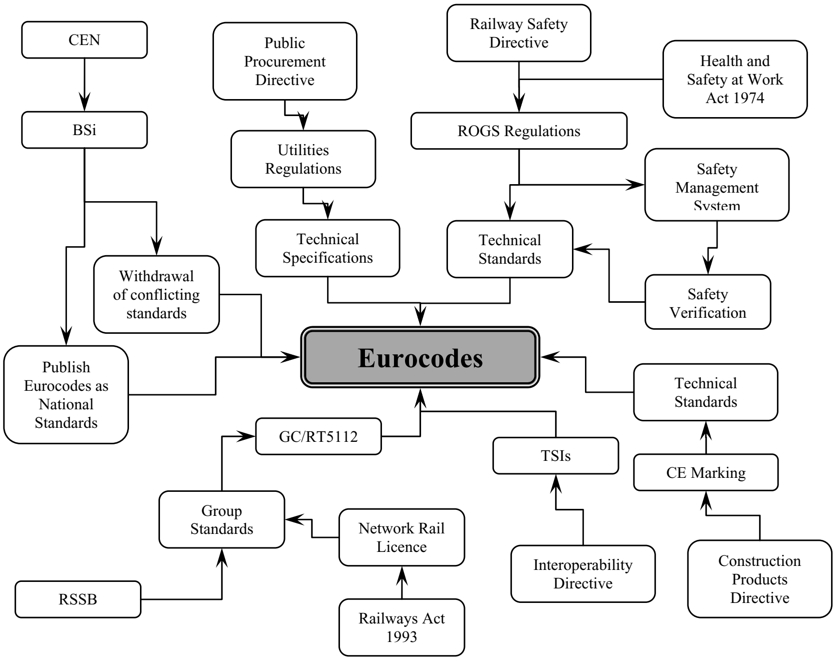

Figure 1. UK Eurocode Requirement Diagram

Implementation Strategy

The organisational structure in NR separates the functions dealing with standards & policy development from works delivery. The Engineering team are part of Asset Management and develop polices and standards to be applied to the infrastructure. The delivery teams for both projects and maintenance, interpret the policy and standards to support the delivery of works to the infrastructure.

NR engineers are engaged in the national standards development process and have representation on industry groups. Contact with other key bodies such as the Highways Agency and RSSB has contributed to the knowledge sharing across the industry. The requirements to implement the Eurocodes into the day to day business processes in NR have been well understood but this knowledge was held by a few key individuals.

In mid 2008 a formal steering group was established within NR to guide the implementation of the Eurocodes into the business and to draw together the knowledge already held. The group was made up of representation from project delivery teams, asset management, legal and procurement. This group identified three key areas of focus:

1. Update the Eurocode policy statement including application and timescales

2. Review of the company standards and documentation which may be affected
3. Develop an understanding of the training requirements and how to deliver it

It should be noted at this stage that the application of Eurocodes to NR's business has been carried out holistically and whilst references in this paper are specifically focussed on bridges, the remainder of the asset groups have been dealt with in parallel and under the same policy and processes.

Policy development

Commercial considerations

The main driver for the development and introduction of Eurocodes is to remove technical barriers to trade. Commercially NR had to understand how and when they would adopt Eurocodes as the preferred methods of design. Three strands were affecting the decision:

1. Application of the PDD to NR
2. Timing of the publication of the Eurocodes and UK National Annexe's
3. Withdrawal of conflicting British Standards

From the earlier section of this paper, the steering group worked through the legislation and standards requirements to the application. The date for the implementation of Eurocodes and consequent withdrawal of conflicting standards was held by BSi to be 31st March 2010.

The legislation refers to 'commercial activity' and so this became the centre of discussion. The interpretation could have significant impact on the training requirements depending on whether this was deemed to be the issue of a pre-qualification document or actual contract award. There could be considerable time between the two stages of procurement. Internal legal advice clarified that commercial activity commences when invitations to tender (ITT) are issued. Therefore all ITT's issued on or after 1st April 2010 would require all technical specifications to refer to Eurocodes as the basis of design. This gave a focus on the timeline for full implementation and allowed a training programme to be developed.

Letter of Instruction

An initial advice note was issued internally in 2007 in response to queries raised by some long lead projects. It was cascaded through the engineering briefing chain to all internal disciplines/functions. The document set out a broad strategy for implementation, listed key milestone dates and gave some background to the Eurocodes and their document structure. This initial advice note was compiled by members of the steering group, albeit twelve months in advance of the steering group being formally convened.

The first draft of the NR Policy Statement 'The adoption of Structural Eurocodes for the design of Civil Engineering Works' was produced in early 2009 and was the output of the steering group review of the 2007 advice note. The policy reflected the updated publishing programme of both Eurocodes and the respective UK National Annexes (NA). The shift from advice note to policy statement gave the document more authority. The main revision was to move the focus away from strategy and to provide direction on how to apply Eurocodes to NR infrastructure. This policy document underwent a number of revisions and further issues to capture the continuing changes, increased understanding and updates of publication schedules throughout the transition period.

By May 2010 version 4 of the policy had been issued internally. Communication with suppliers regarding the application of Eurocodes was still on a project by project basis. It was accepted by the steering group that the information contained within version 4 would not require any major subsequent revision. This was due mainly to Eurocodes and the UK NA's referenced in the policy being fully published.

The purpose of the policy document was to provide clarification on when, what and how areas of the delivery process would be affected by Eurocodes. The steering group required a mechanism to ensure compliance with the details contained within the policy document. In the longer term the details would be captured in the relevant revised company standards or new standards if appropriate. Due to the timescales associated with normal NR standard development process, a short term solution was required. This information also needed to be shared with the supply chain.

The existing NR standards structure allows for the publication of a letter of instruction in certain circumstances, particularly if there is a safety related issue or, in this instance, a time critical non-safety issue, that cannot readily be dealt with via the usual procedures or non-compliance. It was this route which was selected for issue of the policy. Letter of Instruction, NR/BS/LI/187: Application of Structural Eurocodes[24], which was issued on the 16th June 2010 and applies to standard NR/CS/CIV/044: Managing Structures Works[25]. This is a short term solution to mandate policy until NR standards are either revised or a new standard is written to specifically cover the use of Eurocodes.

In summary, the Letter of Instruction covers the following:

1. The scope shall include all works undertaken on NR owned or managed structures
2. The suite of Structural Eurocodes, including their relevant UK NA's, shall be used (wherever appropriate) for the design of (a) new structures, and (b) replacement parts of structures (such as a completely new bridge deck). For (b), the designer shall check the consequences and the compatibility of the design on other parts of the structure (such as the substructure) that have not been designed to the Eurocodes.
3. An addendum to the Approval in Principle submission shall be submitted where the original submission for design was based on the UK national Codes of Practice, but detailed design is to be undertaken using the Eurocodes.
4. Details should be provided of how Eurocodes are to be applied to special construction works
5. Details should be provided of how the requirements of BS EN 1990:2002 are to be applied to other materials such as Fibre Reinforced Plastic
6. Unless the Eurocodes were used to design a structure they should not be used to determine or confirm the safe load capacity of that structure and, from this, determine the strengthening requirements for that structure.
7. Where a design has been completed using the UK national Codes of Practice, the design check and verification of the construction works shall be based on those Codes and/or other compatible standards.

This is a high level summary and reference should be made to the full text of the letter for application purposes.

Standards and documentation review

As a consequence of the adoption of Eurocodes, NR standards and processes required review to identify and update references, to align with the different approach taken in design philosophy by the Eurocodes, and to ensure asset specific standards reflect the relevant information contained within the Eurocodes. The changes to standards and processes are many and varied. They range from simple references through to full standard re-writes. For example, standards such as NR/L3/CIV/140 – Model Clauses for Civil Engineering Works[26], are by nature linked to British Standards and therefore require updating as a result of the introduction of the Eurocodes and the subsequent withdrawal of British Standards. For instance, Section 90 – Steelwork to BS5400 is in the process of being updated but read:

SECTION 90	**STEELWORK (BS 5400)**
Application of standards and codes of practice	Steelwork including handrailing, walkways and bearings forming part of the bridge steelwork fabrication shall comply with BS 5400: Part 6 as qualified or amended by this and the following Clauses.

Figure 2. Extract from Model Clauses – Structural Steelwork

Other standards require significant alteration such as NR/L3/CIV/071 – Geotechnical Design[27]. This is now on issue two and incorporates reference to the Eurocode parts and relevant Non-Contradictory Complementary Information (NCCI), e.g. CIRIA Report C580 – Embedded Retaining Walls[28].

Technical approval

The Eurocodes adopt a different approach to design and, as such, the NR technical approval process (NR/SP/CIV/003 – Technical Approval of Design, Construction and Maintenance of Civil Engineering Infrastructure[29]) needed review. A revised technical approval schedule has been developed as a result to enable it to capture the additional information from the design process.

The steering group received representation from some projects planning to issue design and build (D&B) tenders post 1st April 2010. These projects had an existing Form A (Approval in Principle) that had been prepared to British Standards which formed part of the technical specifications. Given the timing, the detailed design would have to apply Eurocodes as the basis of design. Following consideration of the options and risks the steering group concluded that the technical specification should require the successful D&B contractor to issue a Form A Addendum that would provide any changed information and allow detailed design to commence.

Client choice and options

One of the differences in the design approach between British Standards and Eurocodes is the introduction of client or project specific choice and options found within the Eurocodes and

NAs. Suppliers require direction or guidance from NR on associated requirements, acceptable approaches and how decisions should be documented. Supporting documentation that complements the Eurocodes (NCCI) also requires a level of acceptance by NR and subsequently, this decision has to be cascaded to suppliers as for use in the design of NR structures.

NR has developed a suite of standard designs for civil engineering works[30] under BS design codes. The opportunity existed to verify the underbridge designs to Eurocodes and to compare the outcome against the original. NR current suite of standard designs for underbridges include D, E, U & Z-type, Box Girder and Con-Arch. Mott MacDonald undertook this work and to date the D, E, U and Z-type have been checked using Eurocodes.

To build upon the knowledge gained though this process, NR sought support from Parsons Brinckerhoff (PB) to help identify and provide technical guidance on how to deal with choice and options. This work was to build upon the findings of the standard design work referenced above and other research work undertaken by NR and by RSSB[31][32]. Each separate Eurocode and supporting National Annex has been reviewed and the options and choices have been identified. This has led to the development of a series of tables which summarise where options and choices are available and set out the relevant NR requirements.

Training programme

A strategy was required to deliver training to all of NR's civil engineering professionals on the new suite of Eurocodes. The initial concept was to undertake three levels of training:

1. Awareness of the Eurocodes for all project and engineering staff
2. Application training for both project engineers reviewing designs and engineering staff setting remits and specifications
3. User training for those engaged directly in design activity.

Awareness Briefing would give all staff an understanding of the purpose of the codes, their general structure, statutory environment and new terminology to be adopted. This briefing was to be a short electronic presentation for general cascade throughout the company briefing chain.

Application training was to be aimed at those engineers directly involved in specifying engineering works and those managing the technical approval process. This would deliver a good understanding of the scope, structure and content of the new Eurocodes.

User training would be aimed at engineers involved in the preparation and management of structural designs. From this training group there would be an opportunity that certain trained staff could become 'technical champions' who would develop and maintain an appropriate level of knowledge.

The steering group reviewed the internal expertise in NR and concluded that there was in-sufficient resource with experience in using the new codes to conduct in-house training without impacting the day to day delivery of the business. Therefore, NR would need to approach the external supply market to deliver the training.

The group needed to develop a training strategy to provide best value. Consideration needed to be given to the volume of training required, the time away from the workplace and the timescale for delivery. Following a review of the course availability in the market, consideration was given to procuring a supplier to conduct the training on NR premises. A business case was prepared to support the initiative and, following the outcome of further discussions with suppliers, a decision was made to approach PB to develop and deliver the training.

The business case compared the costs of developing and delivering a bespoke training course at an internal venue against the cost of purchasing a training course from an external training provider. Most courses available from external sources were either single day appreciation style courses or longer, material specific courses. The balance was wrong for a client organisation. The approach taken by NR was to develop asset based training rather than material specific. This would give the NR engineers a course directly related to the asset they were dealing with. The business case predicted a saving of 25% given the volume of staff training required. NR made budget provision and a phased approach to training was developed. Phase 1 would include course material development and the delivery of a pilot course for bridge design. Phase 2 would consist of refining course material using feedback from pilot course and subsequent delivery of a NR specific course.

The user training was removed from the national training programme given that there are a relatively small number of engineers in the design teams. The decision was made to source this detailed training from the market as it was likely that this training would need to be material based rather than by asset type.

Three NR engineers attended an internal PB course on the Design of Bridges to Eurocodes. The aim of this exercise was to gather feedback on the application of the existing course material to NR bridge design. The feedback was used to develop a pilot course for an initial round of training. This revised course was then delivered to two groups of NR engineers, twenty in total. Feedback was gathered from these two pilot courses to further develop the NR specific elements of the course.

In parallel with the development of the bridge design course, courses for buildings and geotechnical were being developed for roll out across the engineering teams. The business case, funding and delivery routes were all shared across the asset types.

Training is now being delivered by PB at NR training centres throughout the UK. All courses are modular and of three days duration with the first day being standard on all three courses. This allows delegates attending multiple courses for different asset types to attend days two and three to minimise the disruption in the workplace.

The main aim of the training is to highlight the different approach that the Eurocodes take to design. The training explains the document structure behind the Eurocodes, the role of NA’s, NCCI and BSi Published Documents. It also explains the difference between the Eurocodes, Product and Execution Standards and how they relate to one another. The training aims to equip the engineers to understand the nature of queries they may receive through the design process and to be able to research an answer.

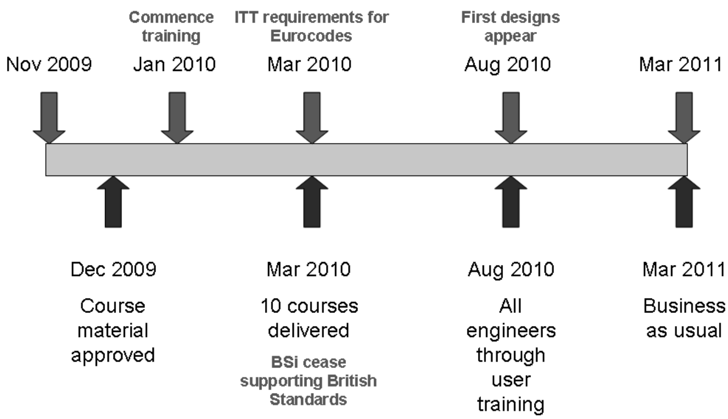

Figure 3. Training timeline current at Nov 2009

The timeline shown in Figure 3 indicates the intent to have the majority of engineers trained by late 2010. On the basis that ITT's would be issued post 1st April 2010, given a tender period and initial design period, it was anticipated that the first designs would be delivered around the middle of 2010. The training programme was developed to match this. Training too early ran the risk of knowledge being unused and forgotten, training too late gave consequential risk to the business delivery.

The current cost analysis against numbers trained is returning 22% saving across the training programme. Predictions for year end in March 2011 should indicate a saving in excess of 30%. However, benefits of this approach are not purely financial. The training courses are bespoke to NR and deal with the specifics of railway assets. There is also the benefit of consistency across the training regime with all engineers receiving the same training.

Conclusions

NR implementation of Eurocodes has been undertaken smoothly by utilising the support of the supply chain. Research by others and work undertaken internally has guided the implementation strategy and informed policy. There is much more to do to refine Client choice and options and work is ongoing in this area as knowledge over the application of the Eurocodes develops.

The delivery of bespoke training has benefitted the business with consistent knowledge levels targeted at the asset types. The feedback of the delegates has been instrumental in developing a course which targets the needs of the business and delivers a quality output.

The scale of change to internal standards and processes should not be under-estimated. Whilst a large proportion of these are simple references, they all require due process to be followed to implement change which is time and resource hungry.

Acknowledgements
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EARLY APPLICATIONS OF EUROCODES IN UK BRIDGE DESIGN PRACTICE

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Abstract
The objective of this paper is to provide bridge engineers with insight into the application process as it applies to the production of designs to the Eurocodes. Switching from BS5400[1] to the EN documents involves a number of changes to the method of implementation of a design and requires a top down revision to design process.

Introduction
In March 2010, the structural Eurocodes became mandatory throughout Europe for public sector bodies. That is, any new design that is prepared for any public sector project must use the Eurocodes in place of national codes. As bridges are almost exclusive the preserve of public sector clients, bridge engineers will be at the vanguard of the change to Eurocodes.

This spotlight may be welcomed by some and viewed as an imposition by others, however, the time for debate on the need for change has long passed. The Eurocodes are now an essential part of the design process for almost all bridge engineers. Almost all? Those who work on projects outside the EU will not be made to change, but may well be working to different codes such as AASHTO[2] or the Australian Bridge Code[3]. For them, the process of application of different code systems is a known process and therefore they should not find difficulty in the application of the Eurocodes.

This paper looks at a number of projects and focuses on how the Eurocodes have been applied to the design of a variety of bridge designs. The application is as much about the process of design as it is a function of the use of the code. It also follows on from an implementation strategy that places the framework for use of the Eurocodes in the heart of the business.

Implementation
Prior to the application of the Eurocodes, it has been necessary to introduce an implementation programme within the design office to ensure that the designers are first familiar with the Eurocodes and secondly have the tools at their disposal to conduct a design efficiently. The process of implementation is not the subject of this paper but it is worth touching on some of the parts of the process as they affect the application of the process.

Perhaps the greatest issue in the application of the Eurocodes has been the need to educate all staff in their use. Bridge design organisations working to BS 5400 will have engineers with varying skill and ability in the use of the code depending on their experience and specific expertise. Those leading the design will have extensive general experience in the application of the relevant parts of BS 5400. The more recently qualified engineers will be developing familiarity with parts of the standard and seeking to learn more on aspects they are yet to tackle. The more specialist aspects of bridge design, such as bearings, barriers or cables will be the domain of the in-house expert who has gathered much more detailed experience and knowledge of the background to his or her specialism over a period of many years.

To an extent, all this needs to be replaced, therefore the implementation phase, where a company plans the switch to Eurocodes, will have required investment and co-ordination to ensure the company is ready to start the design work to the Eurocodes. However, the physics that underpin the principles of the Eurocodes have not changed. The mathematics of calculation is still the same in terms of numerical processes. The primary change is the presentation of the codification of the Eurocodes and some of the subtleties contained therein.

The effectiveness of the training of staff during the implementation programme will be conditional on the amount of real exposure the engineers have had to the application of the codes. In-house lectures and external courses will be of benefit if there has been the opportunity to practice using the code and to find ones way around it unaided. BS5400 was relatively simple to use as one became used to having the full set of design rules in only a few documents. Some of the misjudged criticism of the Eurocodes has been the spread of design information and the removal of a ‘bridge specific’ code. The benefit of the removal of some of these criteria will be the absence of some of the notable contradictions between building codes and bridge codes in the UK.

The period of availability of the national annexes and some of the codes has been much shorter than planned and this will have affected the early birds who sought to get a head start. Apart from notable clients such as the Highways Agency and Network Rail, the vast majority of the clients in the construction sector have adopted a wait and see approach, looking closely at the public sector procurement process to see how it fares. But this is us, the bridge engineers, who are required to take the lead and start the application of the Eurocodes.

It should be borne in mind that the transition to the Eurocodes is a gradual one. Many engineers seemed to have lost sight of this and will have found to their relief that the 1st April 2010 did not see their world ending and that bridge design to BS 5400 has continued on for many current projects. The switch over will be gradual and may take several years to fully implement in a bridge design office as legacy projects to BS 5400 run their course.

Application – Starting Off

Prior to the first application of the Eurocodes, there is the inevitable housekeeping that needs to be performed to facilitate an efficient and error-free design process. The first issue is the sourcing of the Eurocodes and the supporting documentation. For companies with electronic access to the design codes and other Euronorms (the product standards and executions standards), the assembly of the necessary documents is straightforward. For those without, there is a significant investment required for all the relevant parts of the standard. The choice of supporting documentation is important: much work has gone into the Thomas Telford guides and these are recommended as being amongst the best guidance available. It is expected that these conference proceedings will also form a key part of the arsenal.

There is also a welter of other documents that will assist the designer such as the Published Documents (PDs), industry guides from SCI and the Concrete Centre, and technical papers presented at conferences on aspects of the background to the codes. It is a necessary part of the implementation process that these documents are identified and the contents made available to the lead designers, the new designers and the experts.

The second issue is the development of design tools. To the younger engineer, the presence of a computer at every desk with access to frame analysis packages, auto-loaders, finite element tools, non-linear analysis packages, spreadsheets and CAD packages is a far cry from just fifteen years ago. The benefit of these packages is that our ability to analyse in greater detail has improved, as has our ability to seamlessly interface between design and drawing production.

These design tools will have been honed to both the codes of practice in use and the type of structures being assessed. With the advent of the Eurocodes, there will be a need to implement a vast number of changes to software to ensure that the correct traffic loads are used in combination with the correct load factors, that the correct combinations have been used, that the strength calculations have been derived appropriately and compared with the right forces, moments, stresses and deflections. This represents perhaps the greatest investment and requires considerable checking to ensure that automated processes can be relied upon. Software companies do not provide warranties for their product – *caveat emptor* applies in every sense.

The software issue will also affect the use of spreadsheets which are often difficult to police. All engineers armed with a spreadsheet package have software writing opportunities at their disposal and may well have written useful pieces of code to assist them in the calculation of section properties, equivalent section sizes or section capacities: these all need review. Both the implementation and application processes offer companies the chance to rationalise the use of both outsourced and internally developed spreadsheets.

The third issue that affects the early application of the Eurocodes is the simple feel for the calculation process. A criticism of BS 5400 is that there are many empirical formulations for which there is little feels for the physics of the process. This has been addressed in the Eurocodes which avoid such empiricism as much as possible. However, there is still an inevitable degree present and understanding if the output of such a process is correct requires a degree of reworking and testing to ensure that the final design is appropriate.

Experienced designers have a feel for what is acceptable. This may be based on knowledge of where certain parameters should lie for safe and economic design. Where these are not part of the Eurocodes, there will be a need to relearn the new limits. An example of this is the reliance on the use of slenderness limits as opposed to the Euler stress. Engineers familiar with the ‘squash’ limit for struts of around 15 to BS5400 Part 3[4] (for S355 steel) must now also become familiar with the corresponding Euler stress limit of 8875 N/mm^2.

Finally, checking! Bridge design to the Eurocodes should result in broadly the same bridge being designed as it would to BS 5400. Improvements to conservative formulations in BS 5400 will bring economy to some elements, but in the main, the experienced engineer leading the team will recognise the merit of the final design. This simple ‘rough’ check of proportion needs to be backed up with a thorough check of the details. Here the process of the first application of the Eurocodes will involve additional expense.

The process of independent design checking has been enshrined in bridge design for nearly forty years following the collapses of the box girder bridges in the early 1970s. The “Category III” check described in BD 2 [5] remains part of the design process but should never be viewed by designers as a substitute for their own independent checking in-house. There is a degree of comfort engendered by the external check but this must not be used as a crutch by the designer in the change-over to Eurocodes. It will be tempting to use BS5400 as a parallel check, which may in the first instances be appropriate. The benefit here is that it will be a completely independent design approach that will provide a degree of comparative checking. But what to do when BS 5400 offers a marginal “fail” and the Eurocode a “pass”? Simply increasing the section size to suit completely undermines the economy achieved through the Eurocode. The check process should also follow the Eurocode methodology but with critical review of amended spreadsheets, software and other design tools. It takes time and effective review to de-bug software; investment here is vital.

Application – Getting Around to It

Now that the date for withdrawal of British Standards with the same scope as Eurocodes has passed, any new bridge for public sector clients “should” be designed to the Eurocodes. (The subject of when Eurocodes “must” be employed could be the subject of a separate paper since it is affected by European procurement Directives and UK implementing Regulations, which set thresholds for their mandatory use on projects, and also the Construction Product Directive, which sets an expectation that Eurocodes will be used as the default means of demonstrating structural adequacy). Therefore, any bridge designer embarking on a new bridge design in the UK should be doing it to the Eurocodes and will be gaining first-hand experience at the process and may view this paper with wry and hopefully mutual appreciation of the nuances involved.

The current economic climate has seen a scaling back of new design work and so the process may be slowed for some. It is a worthwhile exercise in such circumstances to carry out a pilot design of perhaps an existing project to ensure that the Eurocode design process is understood rather than wait for the opportunity to arise.

The requirement for design to Eurocodes will be slow at first and only require the direct involvement of a few engineers. The selection of these engineers should be guided by ability and familiarity; experience has shown that it is as much availability that governs who does the work.

Application – Experience Gained

The design process itself does not change with the application of the Eurocodes. The first step in applying the Eurocodes is the recognition that engineering design is not simply a process of calculation; it is a far wider process that touches on art, science, people, construction and commerce. Recognition of this does reduce the perception of massive change and puts the use of the new codes firmly into perspective.

Early application of the Eurocodes by the authors has shown that there are inevitably areas where designers feel less confident initially when they approach a design. This is no different to the designer faced with the use of BS 5400 for the first time. The benefit we have found is that the designers taking up the Eurocodes for the first time do so from a standpoint of experience in design and knowledge of bridge behaviour gained from other projects. There is a desire to understand the provisions behind the code, to compare them with the equivalent in BS 5400 and to understand why they differ. Some comfort is gained when the same codified clauses appear in both codes and gradually there is a desire to move forwards and understand the process in greater depth.

Once the talking, conferences, seminars and initial training courses are over, the real learning curve will begin as the detailed designers pick up the mantle and attempt to deliver complete bridge designs to the Eurocodes. Possibly exacerbated by the current economic climate, 21st century commercial pressures will certainly still require that designers deliver their detailed design drawings and specifications within the same timescales and budgets that would be expected using more familiar BS 5400 codes and procedures. Whilst the initial training courses will have given the designer a ‘feel’ for where to start, it must be appreciated that there is no direct substitute for familiarization as applying the new codes in the design of a real structure. Current experience has shown that, even with experienced designers roughly familiar with the principles of the Eurocodes, more time is still needed to deliver a Eurocode design compared to a BS 5400 design on account of the greater need for checking the less familiar calculation procedures. In addition, the new technical approval requirements covered in BD 2 and BD 100 will also require designers to go through a considerable learning curve to understand what is now required from them to complete a ‘Eurocode AIP’. Whilst the above learning curves are being crossed, the resulting extra burdens on designers will need careful management within design organizations if current commercial realities dictate that no commensurate adjustments can be made to design timescales and budgets.

Frustration can arise when a simple process in BS 5400 is apparently more complicated in the Eurocode, requiring greater investigation for what may be little change to the output variable from that obtained in the British Standard. The calculation of shape limits of stiffeners to EN 1993-1-5 is one such example; BS5400 gave simple dimensional limits while EN 1993-1-5[6] requires a more lengthy calculation of section properties or even critical stresses for stiffeners with appreciable warping stiffness, such as “T” stiffeners. Misgivings can also arise when the process is much shorter, suggesting that the rigour of BS 5400 was not required. These are inevitable reactions, but ones that can be addressed by reference back to the source material to understand the reason for the changes.

Case study 1 – Redhayes Bridge

This landmark footbridge features a steel arch spanning across the M5 Motorway near Exeter. The bridge was designed for Devon County Council by Parsons Brinckerhoff with a category 3 check by Atkins. The Highways Agency acted as the Technical Approval Authority. Even though the design commenced prior to the 1st April 2010 mandatory switchover date, it was agreed to use Eurocodes throughout the design and specification process, a decision fully supported by all parties.

The first major challenge was to compile the Approval In Principle (AIP). This is never a trivial process for any structure but for Eurocodes additional efforts are needed. The assistance of the Highways Agency was essential here for providing draft versions of the revised BD2 and new BD100[7] documents, standards which give the AIP proforma and specific Eurocode implementation requirements respectively. The latter includes a long list where project specific decisions need to be made. These arise from the many instances where Eurocodes allow different approaches to be taken, or require project specific parameters to be determined, and many of these need to be declared in the AIP. Therefore the AIP process took considerably longer than had a BS 5400 design been used.

At the start of the design process, the risks of using unfamiliar standards were clearly recognised. In order to address the risks three specific actions were taken. The first was to write design method statements for each design calculation. These gave a clause by clause procedure for every design check of every element in the structure. The second was to set up a peer review team. The peer review team carefully checked all the design method statements, undertook parallel calculations for some specific elements of the structure, and had an overview of the drawings. The third action was to ensure the design was supervised by experienced engineers, who would detect any rogue results.

The structure was modelled as a space frame using the programme Lusas[8]. This allowed advanced dynamic analyses to be undertaken, and the possible use of second order analysis. However, tests on the model showed that differences between first and second order results were small and as a result first order analysis was used for the design. This would have been no different had BS 5400 been used although the Eurocodes appear to direct the engineer towards second order analysis much more strongly than BS 5400.

One notable difference between BS 5400 and Eurocode is in the approach to lateral torsional buckling. BS 5400-3 generally has an equation-based approach to lateral torsional buckling, which is not present in EN 1993. PD 6695-2[9] reintroduces some of the equation based approaches in a Eurocode compatible format, but the case of lateral torsional buckling of a box girder, which was addressed in BS 5400-3 is not present. To fully investigate lateral torsional buckling of the main arches, it would have been necessary to build a detailed finite element "shell" model of the arches. The design team did not want to spend time building an unnecessary model, so a rare exception was made and BS 5400-3 rules were used by the designers to satisfy themselves that lateral torsional buckling would not govern the arch resistance. It is possible to convert slendernesses obtained from BS5400 Part 3 to slendernesses in Eurocode 3 but this is not generally recommended as the BS5400 slendernesses can be empirical.

Rather than producing hand calculations, section resistances were determined using MathCAD[10] scripts. Although these had to be written from scratch, the production of Eurocode calculations was considered to be no more complex than BS 5400 calculations, and in many cases the calculations would have been almost identical. The only exception to this was found in the control of early thermal cracking, where as a result of the latest research, the Eurocode approach is more sophisticated than its predecessors in BS 5400 or BD 28/87[11].

Case Study 2 –Cable Stayed Road Bridge Design

Perhaps at the other end of the spectrum, this case study centres around a tender design carried out for a major cable stayed road bridge, currently under tender. The structure has a steel box with an orthotropic stiffened deck. The design was carried out using EN 1993-1-5, which requires a comprehensive understanding of stiffened panel behaviour. There are two options available to the designer, the reduced area method and the reduced stress method. However, application of the code rules has shown that the designer needs to extend their knowledge considerably beyond the code in order to be able to apply the rules. The situation is compounded by the fact there is little or no guidance available even within the wealth of various design guides and as such this is likely to remain an area that all but a few designers or design organisations will be discouraged to use.

Case Study 3 - Compiegne Bridge

This project, designed by Flint & Neill, was selected as an early design to Eurocodes by the French client. The challenge here was to make use of the French National Annex in place of the British NA. Not only did this involve linguistic challenges, it also offered additional challenges of understanding the reasons behind the selection of French Nationally Determined Parameters. Advantages of designing to the EN were clear for the proportioning of the slender steel arch that is the main support for this highway bridge. EN 1993 has provided a clear process to follow: in this case, the selected design approach involved prescribing an equivalent imperfection (intended to represent the effect of geometric imperfections + the effects of locked in stresses). Thereafter a large displacement analysis was used to assess the load carrying capacity of the arch. It is interesting to note that when applying BS 5400, a very similar approach was typically used by F&N when analysing very slender elements in order to optimise designs albeit that this effectively represented a departure from standard. By providing a clear codified process, EN 1993 provides a much more accessible design tool to many more designers.

The structure has an orthotropic steel deck. EN 1993 provides state of the art guidance in terms of the design and construction of orthotropic steel decks. Such procedures of guidance were not available to designers directly within BS5400 thus requiring departures and extensive consultations when designing such elements. The methods and guidance within EN 1993 have been developed in part to provide a ready tool for engineers and thus altogether improving design efficiency. Ultimately this will expand the options available to designers as a whole thus leading to more imaginative design and ultimately more cost effective design solutions.

Case Study 4 – St Helen's Footbridge

This 45m span arched footbridge is currently under design development working towards target cost. Whilst the learning curves for the technical content of the Eurocodes are well documented and known, this job has shown there is an even larger learning curve regarding the new Technical Approval procedures as required in the new versions of BD 2 and BD 100. This additional learning curve was not anticipated. Unfortunately, the AIP is now more of a 'Detailed Design and Specification Report' – most of which can only be completed upon completion of the detailed design for an unfamiliar, bespoke structure. Perhaps this will change as the UK develops greater experience.

However, the wider scope of the Eurocodes is more suited to the design of this complex footbridge compared to BS 5400. The trade off has been the additional effort at AIP stage which is seen as a necessary evil in that it enables the client to be sure of the methodology of design and which allows the experienced client to be comfortable with the design approach adopted. This is important where such significant choices as the use of elasto-plastic non-linear analysis are used to justify designs. The use of such analytical techniques are clearly advantageous in ensuring economy but must be matched by a robust understanding of the background to the design standards and a very strong knowledge of the pitfalls of such an advanced analysis.

Conclusions

The application of the structural Eurocodes has, in the main, been a positive experience. The learning curve on the additional effort required to train staff is not insignificant and must be fully appreciated before embarking on the use of the code for the first time.

It is clear that the AIP process requires a lot more development work and detailed design effort than was necessary under BS5400. This can be a little frustrating when compared with the previous method of working and that it is an 'In Principle' document. However, the completed document does represent a clear statement of design purpose.

The widened scope of the codes is welcomed; the designer is generally provided with a greater arsenal of tools without having to seek recourse for departures from standards, which will invariably be welcomed by clients. The ability to use advanced tools is also welcomed but there must be some recompense for the additional cost involved both in the design and the checking; it would be nice to think that some of the savings gained in the fabrication and construction were set aside for this work!

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SESSION 1-2:

EN 1990 – BASIS OF DESIGN

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UNDERSTANDING KEY CONCEPTS OF EN 1990

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Abstract

BS EN 1990:2002 establishes principles and requirements for the safety, serviceability and durability of structures, and describes the basis for their design and verification. It is a key document and one that all bridge designers using the Eurocodes will need to be familiar with. In particular, it explains how actions should be combined and contains partial factors on these actions and ψ factors to be used in evaluating their design effects. Perhaps most importantly, however, it establishes the overall framework of principles used by the other Eurocode parts. The paper provides an overview of BS EN 1990:2002 drawing out important issues with particular emphasis on those that represent a change from past UK bridge design practice. In doing so, six key new concepts are identified that bridge designers should understand. A summary of each of these concepts is provided

Notation and definitions

The notation and definitions used in this paper are as BS EN 1990:2002.

Introduction

BS EN 1990:2002 was the first of the Eurocodes published, and it is frequently referred to as the 'head' Eurocode. This is because BS EN 1990:2002 essentially serves a dual role, and understanding this fact is very helpful in understanding the Standard.

As would be expected, BS EN 1990:2002 sets out principles and requirements to be applied by designers. In addition, it also establishes the overall framework of tools and principles used by the drafters of the other Eurocode parts.

As a result, BS EN 1990:2002 includes some very general statements, such as clause **2.1(2)P** which states that a structure 'shall be designed to have adequate structural resistance, serviceability, and durability'. Whilst this is clearly an entirely sensible statement, BS EN 1990:2002 gives little guidance on how this should actually be done. Once designers are familiar with the full Eurocode suite this is not a problem, because the means of fulfilling such general principles are given in the other Eurocode parts.

It is sometimes helpful to think of BS EN 1990:2002 as a toolbox – providing the tools that are then used by the other Eurocode parts. This can make reading EN 1990 in isolation rather tricky as it is not always immediately clear how the tools it creates are to be deployed. This paper aims to help with that challenge.

It does so by providing an overview of BS EN 1990:2002 following its structure and drawing out important issues with particular emphasis on those that represent a change from past UK design practice. In doing so, six key new concepts are identified that bridge designers should understand. These are: design situations; reversible and irreversible serviceability limit states; representative values of variable actions; the six different ultimate limit states; the single source principle; and the five general expressions for the combination of actions. A summary of each of these concepts is provided in Appendix A.

The background to the UK National Annex to BS EN 1990:2002 is described by Lane et al[1].

Section 1 – General

Section 1 of BS EN 1990:2002 sets out its scope and assumptions. It also contains definitions and notation. It is noteworthy that the scope of BS EN 1990:2002 includes 'structural design of civil engineering works, including execution and temporary structures', i.e. temporary works (clause **1.1(2)**), and also that it includes 'the structural appraisal of existing construction, in developing the design of repairs and alterations or in assessing changes of use' (clause **1.1(4)**). However, there is an important note below this latter clause that explains that 'additional or amended provisions' may be required for this purpose, which enables the UK to maintain the use of existing assessment standards for bridges.

It is also worth noting that the assumptions given in clause **1.3(2)** are quite onerous and impact the designer, contractor and client. They include requirements for competency and quality control.

Section 2 – Requirements

Section 2 of BS EN 1990:2002 sets out basic requirements, and also general requirements for reliability management, design working life, durability and quality management.

The basic requirements include the three stated in clause **2.1(2)P**. These are that the structure should be designed to have adequate structural resistance, serviceability, and durability. There is, however, effectively a fourth basic requirement embodied in clause **2.1(4)P** which states that a structure 'shall be designed and executed in such a way that it will not be damaged by events…to an extent disproportionate to the original cause'. This clause requires structures to *robust*, and designers should be very mindful of this fundamental requirement, particularly when designing structures with complicated structural forms, when using brittle (or quasi-brittle) materials, components or connections, and in structures with limited redundancy (*i.e.* without alternative load paths).

Section 3 – Principles of limit state design

Section 3 of BS EN 1990:2002 sets of general principles of limit state design, addressing both ultimate and serviceability limit states.

Design Situations

BS EN 1990:2002, **3.2**, introduces the concept of design situations. This is the first of the six key new concepts summarised in Appendix A. Design situations are circumstances (sets of physical conditions) that the structure might experience during its life. As explained in clause **3.2(3)P**, the design situations taken into account in the design, 'shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure'. Although it is important that the designer satisfies him or herself that this principle has been followed, in general, the design situations that need to be considered in bridge design are addressed through the requirements for actions in the various parts of BS EN 1991, and in the requirements given in the other relevant Eurocode parts, depending on the materials used and form of construction.

The real usefulness of the concept of design situations, however, lies in the way in which they are classified. Design situations are drawn together into families that share common characteristics. These categories or families are called persistent, transient, accidental and seismic design situations. Since seismic design is rarely required in UK, it is the persistent, transient and accidental categorisations that are of most interest.

The value of these categorisations is that they recognise that the design requirements for the different families may be different. In practice, the distinction between persistent and transient design situations is rather subtle, but the treatment of accidental design situations is quite different.

Persistent design situations refer to conditions of 'normal use' (clause **3.2(2)P**) . The word 'persistent' is used because the structure will be in this configuration with the potential to experience one of this family of design situations for an extended period of time, in fact, typically for most of its design working life.

Transient design situations refer to temporary conditions when a structure is itself in some special configuration for a period of time, such as during execution or maintenance. An important distinction between persistent and transient design situations therefore stems from the different duration of exposure, so that for example, for transient design situations it can be reasonable to use reduced wind and thermal actions because of the shorter duration of the design situation.

Accidental design situations refer to exceptional conditions in which there is typically some extreme accidental event, such as a vehicle impact with a bridge pier or superstructure. The important distinction with accidental design situations is that, because they are so unlikely to occur in practice, some degree of damage to a structure can typically be accepted.

In bridge design identifying whether a design situation is accidental, transient or persistent is usually straightforward. If the situation involves an accidental action then it is an accidental design situation. If not, and the structure is itself in some special configuration for a short period of time, then it is a transient design situation. And if it is not a transient design situation, it will be a persistent design situation.

Ultimate limit states

Ultimate limit states are defined in BS EN 1990:2002 as limit states that concern the safety of people, and/or the safety of the structure (see clause **3.3(1)P**). This definition is similar to past UK bridge design practice. However, as discussed later, a distinction is made between six different specific ultimate limits.

Serviceability limit states

Ultimate limit states are defined in BS EN 1990:2002 as limit states that concern the functioning of the structure or structural members under normal use; the comfort of people; or the appearance of the construction works (see clause **3.4(1)P**). Again this definition is similar to past UK bridge design practice.

However, BS EN 1990:2002 then introduces in a concept that was not expressed explicitly in past UK practice, when in clause **3.4(2)P** it states that a distinction shall be made between reversible and irreversible serviceability limit states. This is the second of the six key new concepts. Of the six, it is the one that perhaps has the least direct impact on bridge design, but it plays an important role in understanding the different combinations of actions defined for serviceability limit state verifications as discussed later (as the sixth key new concept).

The concept of reversible and irreversible serviceability limit states is perhaps best understood considering the case of a simply supported reinforced concrete beam with a point load at mid-span. As the load applied to the beam is increased its deflection will also increase. At some point this deflection may exceed a serviceability criterion. Whilst this is not an event that the designer would wish to occur (too frequently), provided the beam remains elastic the beam will return to an acceptable deflection when the load is reduced *i.e.* it is *reversible* condition. The situation is rather different if the steel reinforcement yields when the load is further increased. Yielding of the reinforcement is another serviceability criterion and if it occurs it will mean that some permanent damage will be done to the beam; it will not return to its original position when it is unloaded and cracks will remain *i.e.* it is *irreversible* condition. Clearly, this is a more serious situation than the reversible condition.

Thus, it can be seen that not all serviceability limit states are of equal concern. Those which are reversible are of less concern than irreversible once. Differentiating between reversible and irreversible serviceability limit states is useful because it enables a different probability of exceedence to be applied to each. As will be seen later, this can be done by using different combinations of actions for reversible and irreversible serviceability limit states.

Section 4 – Basic variables

Section 4 of BS EN 1990 covers the three sets of basic variables considered in structural design, *viz*: actions, material properties and geometry. Here the treatment of actions and material properties will be discussed.

Actions

It is appropriate first to note the use of the term actions in this context. In past UK practice, the term loads has traditionally been used, and in fact it remains an entirely valid term in a Eurocode context. However, in the Eurocodes the term loads is used to refer to a set of forces applied to a structure or the ground (*i.e.* direct actions). The term action is used more generically to mean both loads and also imposed deformations or accelerations, such as those due to thermal movements or earthquakes (*i.e.* indirect actions). In many ways, the use of the term actions addresses an ambiguity in the way the term load has been used in the past.

Actions are classified by their variation in time as either (see clause **4.1.1(1)P**):

i. *permanent actions* (denoted G), *e.g.* self-weight of structures, road surfacing and indirect actions such as uneven settlements;
ii. *variable actions* (denoted Q), *e.g.* traffic load, wind and thermal actions; or,
iii. *accidental actions* (denoted A), *e.g.* impact from vehicles.

It will be sensible for designers to become familiar with this terminology, rather than using the terms dead and live load. Likewise, it will be advisable to reserve the words persistent and transient for design situations. Referring to a transient load in a Eurocode context is potentially rather confusing since it mixes the terminology for actions and design situations.

For permanent actions, BS EN 1990:2002,**4.1.2(2)P** explains that their characteristic value should either be taken as a single value, G_k, or if the variability of G cannot be considered as small, as the worst case of an upper value, $G_{k,sup}$, or a lower value, $G_{k,inf}$. Further guidance is provided on where the variability can be considered to be small and specifically, BS EN 1990:2002, **4.1.2(5)** states that the self weight of the structure may be represented by a single value G_k based on mean density and nominal dimensions.

In bridge design, important cases where the variability of G cannot be considered as small are loads due to surfacing and ballast (see BS EN 1991-1-1:2002, **5.2.3**). When the variability in G cannot be considered as small, it is helpful to note that **4.1.2(2)P** does not require upper and lower values of G to be applied to the adverse and relieving areas of the influence surface. Rather, whichever single value gives the worst case is taken throughout

For variable actions, BS EN 1990:2002, **4.1.3** introduces another new concept for UK bridge designers. This is the concept of the four *representative values* of a variable action, and it is the third key new concept, as summarised in Appendix A. As discussed later, these representative values are used in the different combinations of actions.

The four representative values have different probabilities of occurrence. They are called the characteristic, combination, frequent and quasi-permanent values. The characteristic value is the main representative value, and is the value generally specified in the various parts of BS EN 1991. It is a statistically extreme value: in the calibration of the basic highway traffic loading model, LM1, it is a 1000-year return period value (see BS EN 1991-2: 2003, **Table 2.1**); for wind and thermal actions it is generally a 50-year return period value.

The combination value is established by BS EN 1990:2002 to address the reduced likelihood that extreme values of more than one variable action will occur simultaneously. The frequent value of a variable action can be understood as the value that is exceeded 'occasionally, but not too often' – perhaps weekly or monthly. The calibration of the frequent value of LM1 is based on a one week return period. The use of the word frequent here sometimes causes some confusion, since it is essentially a relative term; here it is frequent in relation to the characteristic value. The quasi-permanent value is generally the value that is exceeded most of the time. For traffic loads on bridges and wind actions, the quasi-permanent value is therefore zero.

The four representative values of a variable action are illustrated in Figure 1. The combination, frequent and quasi-permanent values of a variable action are found by multiplying the characteristic value by ψ_0, ψ_1, and ψ_2 respectively. For bridge design, recommended ψ-factors are given in BS EN 1990:2002, **A2.2**. The UK National Annex modifies the values for road bridges and footbridges.

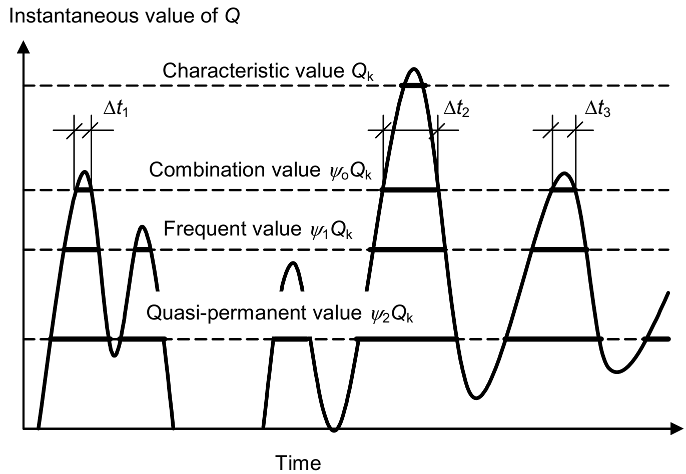


Figure 1. Illustration of four representative values of a variable action

Material and product properties
BS EN 1990:2002, **4.2(1)** explains that properties of materials (including soil and rock) should be represented by characteristic values. It also states that when a limit state verification is sensitive to the variability of a material property, upper and lower characteristic values of the material property should be taken into account (clause **4.2(2)**). Although it is rare that an upper characteristic material property will govern a design, rather than the lower value that is generally used, there are some important cases in bridge design when it can do so. These include earth pressures applied to integral bridges and other buried structures, where an upper characteristic angle of shearing resistance of the soil can govern.

Section 5 – Structural analysis and design assisted by testing
Section 5 of BS EN 1990:2002 gives general principles and requirements for structural modelling and analysis. These provide the framework for the more detailed treatment included in the various Eurocode material parts.

Section 6 – Limit state design and Annex A2 – Application for bridges
Section 6 of BS EN 1990:2002 describes how the partial factor method is applied in limit state verifications. It provides the overall framework for the applications of the partial factor method, including the way in which actions are combined and partial factors are applied. It is best considered, however, in conjunction with BS EN 1990:2002, **Annex A2** which gives supplementary bridge-specific requirements for establishing combinations of actions (except for fatigue verifications which are typically addressed in the relevant material part), provides ψ-factors and material-independent partial factors, and also gives methods and rules for some material-independent serviceability limit states (*e.g.* vibrations and deformations of rail bridges).

Design values

The design values of action effects are determined accounting for uncertainties in the actions themselves and also uncertainties in the evaluation of effects of actions. Similarly, design values of resistances are determined accounting for uncertainties in material properties and also uncertainties in resistances models.

Strictly this is done by using two partial factors in determining action effects (with one applied to the action and the other to the effect of the action) and two partial factors in determining resistances (with one applied to material properties and the other applied to resistances). These factors are:

Action effects:	γ_f	partial factor for the action which takes account of the possibility of unfavourable deviations of the action values from the representative values
	γ_{Sd}	partial (model) factor taking account of uncertainties in modelling the effects of actions
Resistances:	γ_m	partial factor for the material which takes account of the possible unfavourable deviations of a material from its characteristic value
	γ_{Rd}	partial (model) factor covering uncertainty in the resistance model

The model factors γ_{Sd} and γ_{Rd} are similar to the partial factor γ_{f3} used in past UK bridge design practice (see BS 5400[2]). They are illustrated in Figure 2 (included towards the end of the paper).

Whilst it is quite rational to recognise these four different sources of uncertainty, in practice the application of partial factors is generally simplified in the Eurocodes by combining:

i. γ_f and γ_{Sd} into a single partial factor denoted γ_F (or more specifically γ_Q for variable actions and γ_G for permanent actions), and,
ii. γ_m and γ_{Rd} into a single partial factor denoted γ_M

Values of γ_F and γ_M are given in the relevant Eurocode parts, and their National Annexes, with material-behaviour independent factors (*i.e.* almost all partial factors on actions) given in BS EN 1990:2002, **Annex A2**. Clearly for linear analyses combining the partial factors in this way will not affect the overall result. For non-linear analyses some careful thought is always required concerning the correct application of partial factors (see *e.g.* BS EN 1992-2, **5.7**).

Ultimate limit states

BS EN 1990:2002 and BS EN 1997-1:2004 require *six ultimate limit states* to be explicitly verified where relevant. Although all of these would have been considered in past UK bridge design practice, their explicit identification and treatment is the fourth key new concept, as summarised in Appendix A.

The six ultimate limit states are referred to as EQU, STR, GEO, FAT, UPL and HYD. Three of these (EQU, UPL and HYD) are principally concerned with stability, and three (STR, GEO and FAT) are principally concerned with resistances. Two (Uplift and Hydraulic heave) are only dealt with in BS EN 1997-1:2004 and are rarely relevant in bridge design so will not be considered further here.

The three ultimate limit states principally concerned with resistances, STR, GEO and FAT, cover failure of structural members, failure of the ground and fatigue failure respectively. The EQU ultimate limit state covers the loss of static equilibrium of a structure, although as discussed further below, it has a very important relationship with the single source principle.

The usefulness of explicitly identifying six different ultimate limit states lies in the opportunity it provides to use different criteria and different partial factors in their verification. For example, in EQU verifications the partial factors on actions given in National Annex to BS EN 1990:2002, **Table NA.A2.4(A)** are used; for STR verifications not involving geotechnical actions of resistances, the partial factor in **Table NA.A2.4(B)** are used; and, for STR and GEO verifications involving geotechnical actions of resistances the partial factors in both Table **NA.A2.4(B)** and Table **NA.A2.4(C)** are used. This latter case is discussed by Denton et al[3].

Single source principle

Tables NA.A2.4(A)-(C) give two partial factors for each permanent action: a higher value, denoted, $\gamma_{G,sup}$, to be used when the action is unfavourable; and, a lower value, denoted $\gamma_{G,inf}$, to be used when the action is favourable.

There is, however, a very important note in the UK National Annex to BS EN 1990:2002, **Table NA.A2.4(B)** and **Table NA.A2.4(C)**. This note states that the characteristic values of all permanent actions from one source may be multiplied by $\gamma_{G,sup}$ if the total resulting action effect from this source is unfavourable, and by $\gamma_{G,inf}$ if the total resulting action from this source is favourable. This note is a statement of the *single source principle*, which is the fifth new key concept in Appendix A.

The single source principle is very convenient for designers as it means that it is not necessary to apply different partial factors to the favourable and unfavourable parts of a permanent action arising from a single source such as a continuous bridge deck (*i.e.* to the adverse and relieving areas of the influence surface). Because the note is included in **Table NA.A2.4(B)** and **Table NA.A2.4(C)**, it means that the single source principle may be used in STR and GEO verifications.

There is, however, a risk in applying the single source principle, particularly in conjunction with the single characteristic value for a permanent action allowed by BS EN 1990:2002, **4.1.2(2)P**. This risk arises because the sensitivity of the structure to minor variations in the magnitude or spatial distribution of a permanent action from a single source is not examined, and where such minor variations could lead to collapse it is critical that this is done. The EQU ultimate limit state fulfils this purpose, and the single source principle is not (in fact, must not) be applied at EQU.

In reality, cases where minor variations in the magnitude or spatial distribution of a permanent action from a single source could lead to collapse are rare. They should certainly be very rare in persistent design situation, since if not, it would clearly be questionable whether sufficient robustness is being achieved in designs. Typically, the collapse load of statically indeterminate structures with even very modest ductility will be insensitive to variations in the magnitude or spatial distribution of a permanent action. Cases can, however, be unavoidable in transient design situations, such as during bridge launches or in balanced cantilever construction.

Special cases in the application of EQU

There is recognised issue with the current drafting of the definition of EQU in BS EN 1990:2002, **6.4.1(1)P**. EQU is defined as, 'loss of static equilibrium of the structure or any part of it considered as a rigid body, where (i) minor variations in the value or the spatial distribution of actions from a single source are significant, and (ii) the strengths of construction materials or ground are generally not governing'.

The first part of this definition explains that EQU is concerned with a loss of static equilibrium of the structure or any part of it considered as a rigid body, *i.e.* the formation of a collapse mechanism. It is perhaps questionable whether it needs to be explicitly stated that the structure or any part of it needs to be considered as a 'rigid body', but otherwise the intention is clear. The second part of the definition aligns with key role of EQU to account for the implication of possible minor variations in the value or the spatial distribution of actions from a single source. A difficult arises, however, with the third part of the definition, particularly since it is given as an additional requirement (*i.e.* the word 'and' used) rather than an alternative one.

The difficulty is that there are cases where minor variations in the value or the spatial distribution of actions from a single source could lead to collapse, but the strengths of construction materials or the ground are governing. An example would be the design of a prop to prevent overturning of the deck during balanced cantilever construction. The UK National Annex to BS EN 1990:2002 acknowledges this issue in **Table NA.A2.4(A)** note 9, but is not explicit on how such cases should be treated, explaining that appropriate factors may be determined on a project specific basis.

Although such cases are themselves rather rare, being effectively a special case of a special case, it is valuable to provide some advice on how they should be treated. Firstly, it is clearly crucial (and a necessary part of the EQU limit state) that the single source principle is not applied, *i.e.* that the favourable and unfavourable parts of permanent actions from a single source are modelled and factored separately.

Secondly, applying either the partial factors for permanent actions in **Tables NA.A2.4(A), (B) or (C)** alone will be not appropriate. The partial factors for permanent actions in **Tables NA.A2.4(A)** reflect relative uncertainty in their value and spatial distribution; whereas those partial factors for permanent actions in **Tables NA.A2.4(B) and (C)** reflect overall uncertainty in the magnitude of the action effect.

Generally, it will be appropriate to adopt the following approach where minor variations in the value or the spatial distribution of permanent actions from a single source are significant *and* the strengths of *construction materials* are governing:

i. model the favourable and unfavourable parts of permanent actions from a single source separately
ii. factor the (effects of) unfavourable parts of permanent actions by the product of γ_G* and $\gamma_{G,sup}$ as given in **Table NA.A2.4(A)**
iii. factor the (effects of) favourable parts of permanent actions by the product of γ_G* and $\gamma_{G,inf}$ as given in **Table NA.A2.4(A)**

where γ_G* is either $\gamma_{G,sup}$ or $\gamma_{G,inf}$ as given in **Table NA.A2.4(B)**, whichever is more onerous for the particular verification.

Where minor variations in the value or the spatial distribution of permanent actions from a single source are significant *and* the strength of the *ground* is governing, it is likely to be appropriate to use a similar approach and adjust the **Table NA.A2.4(B)** and **Table NA.A2.4(C)** partial factors is a similar fashion, applying them in conjunction with the partial factors on materials and resistances defined in BS EN 1997-1:2004, **2.4.7.3.4.2** as combination 1 and 2 respectively according to Design Approach 1. (See Denton et al[3] for background on design approaches).

Combinations of actions

BS EN 1990:2002 identifies five general expressions for the *combination of actions* that are used for bridge design in UK. There is a further combination for seismic design, but this is not usually relevant in UK and is not considered here.

Combinations of actions is the sixth key new concept summarised in Appendix A. Their application is discussed in more detail and demonstrated by Stacy et al[4]. They are summarised in Table 1. Each combination of actions has a different statistical likelihood of occurring and they are used for different limit state verifications.

BS EN 1990:2002 expresses the requirement that all actions that can occur simultaneously should be considered together in these combinations of actions (see clause **A2.2.1(1)**). There are, of course, cases where for functional or physical reasons actions cannot occur simultaneously and examples are given in BS EN 1990:2002, **A2.2**. In the case of bridge design, the way in which actions are combined is further simplified by forming traffic loads into groups which are then treated as a single (multi-component) variable action (again, see Stacy et al[4]).

Two combinations of actions are used for ultimate limit state verifications: one is used for persistent and transient design situations and the other for accidental design situations.

Three combinations of actions are used for serviceability limit state verifications. These are called the characteristic combination, the frequent combination and the quasi-permanent combination. The quasi-permanent combination is also used for calculating long-term effects, such as creep. Although not always honoured by the other Eurocode parts, it was the intention of BS EN 1990:2002 that the characteristic combination would generally be used for irreversible serviceability limit state verifications and the less onerous frequent combination would be used for reversible serviceability limit state verifications.

		BS EN 1990 Equ^n (3)	Permanent actions, G_k	Prestress, P	Accidental action	Leading variable action, $Q_{k,1}$		Accompanying variable actions, $Q_{k,j}$ ($j > 1$)	
			$\gamma^{(1)}$	$\gamma^{(1)}$		$\gamma^{(1)}$	$\psi^{(2)}$	$\gamma^{(1)}$	$\psi^{(2)}$
Ultimate limit states	Persistent or transient design situations	6.10	γ_G	γ_P	n/a	γ_Q	1.0	γ_Q	ψ_0
	Accidental design situations	6.11	1.0	1.0	A_d	1.0	$\psi_1^{(4)}$	1.0	ψ_2
Serviceability limit states$^{(6)}$	Characteristic combination	6.14	1.0	1.0	n/a	1.0	1.0	1.0	ψ_0
	Frequent combination	6.15	1.0	1.0	n/a	1.0	ψ_1	1.0	ψ_2
	Quasi-permanent combination$^{(5)}$	6.16	1.0	1.0	n/a	1.0	ψ_2	1.0	ψ_2

Notes:

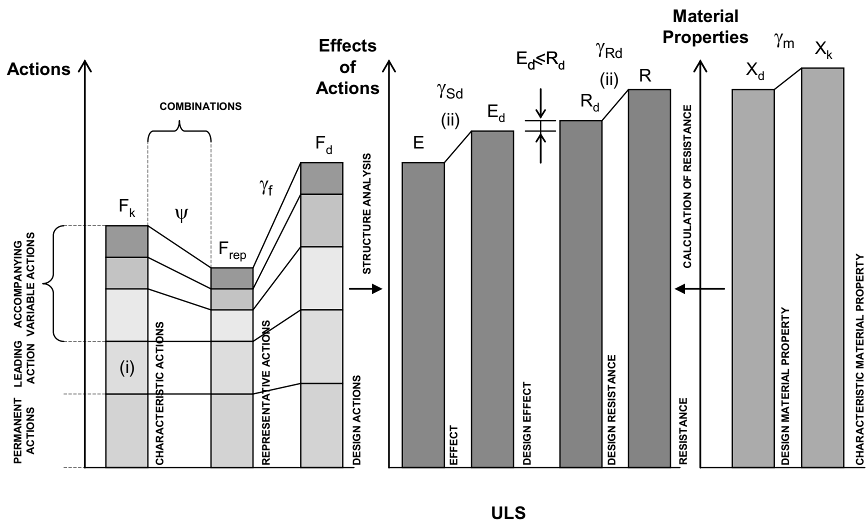
(1) Values of γ are obtained from **Tables NA.A2.4(A)-(C)**
(2) Values of ψ are obtained from **Tables NA.A2.1**, **Table NA.A2.2**, **Table A2.3** for road bridges, footbridges and rail bridges respectively
(3) Expressions 6.10a and 6.10b are not used in bridge design, see **NA.2.3.7.1**
(4) Expression 6.11 allows the use of either or ψ_1 or ψ_2, but ψ_1 is generally used for bridges, See **Table NA.A2.5**. However, see also BS EN 1990:2002, **A2.2.5(3)**.
(5) Also used for long term effects.
(6) Guidance on which combination should be used for specific serviceability limit state verifications is given in the relevant parts of EN 1992 to EN 1999.

Table 1. Combinations of actions used in UK bridge design

Limit state verification

The approach to limit state verification is dependent on the limit state being considered but in all cases is based on ensuring that the relevant effect does not exceed a relevant design value, which may be a resistance, a stabilising action or some serviceability criterion (see BS EN 1990:2002, **6.4.2(1)P**, **6.4.2(3)P** and **6.5.1(1)P**).

As an illustration, the overall approach to the verification of STR ultimate limit state for a persistent or transient design situation is shown in Figure 2. This figure highlights the way in which partial factors and ψ-factors are applied, including the way in which γ_f, γ_{Sd}, γ_m and γ_{Rd} may be used, although as discussed above and indicated in note (ii) they are more generally combined into two partial factors γ_F and γ_M.



(i) Where the action is a traffic load group, ψ factors will have been pre-applied to the non-leading actions within that group

(ii) In many cases, γ_{Sd} may be combined with γ_f and applied as a single factor γ_F to the actions, and γ_{Rd} is combined with γ_m and applied as a single factor γ_M to the material properties.

Figure 2. Verification of STR limit state for persistent or transient design situation

Conclusions

An overview of the key aspects of BS EN 1990:2002 relevant to bridge design has been presented. Six key new concepts have been identified that bridge designers should understand, *viz*:

i. design situations;
ii. reversible and irreversible serviceability limit states;
iii. representative values of variable actions;
iv. six ultimate limit states;
v. single source principle; and,
vi. combinations of actions.

The first five concepts all play a key role in understanding the sixth concept. The category of design situation dictates the combination of actions used for ultimate limit state verifications. The distinction between reversible and irreversible serviceability limit states explains why both the characteristic and frequent combinations of actions are used for serviceability limit state verifications. The four representative values of variable actions play a key role in accounting for the reduced likelihood that extreme values of several variable actions will occur at the same time and in the various combinations of actions having different statistical likelihoods of occurring. The six ultimate limit states and the single source principle dictate how partial factors are applied and the values used for persistent and transient design situations.

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Appendix A – Summary of key new concepts

Key concept summary 1: Design situations

Design situations are categorised as persistent, transient, accidental or seismic. These categorisations draw together families of circumstances or conditions that the structure might experience during its life. Persistent design situations refer to conditions of normal use. As such, for a highway bridge, they will include the passage of heavy vehicles since the ability to carry heavy vehicles is a key functional requirement. Transient design situations refer to circumstances when the structure is itself in some temporary configuration, such as during execution or maintenance. Accidental design situations refer to exceptional circumstances when a structure is experiencing an extreme accidental event.

Key concept summary 2: Reversible and irreversible serviceability limit states

The Eurocodes differentiate between reversible and irreversible serviceability limit states. Irreversible serviceability limit states are of greater concern than reversible serviceability limit states. The acceptable probability of an irreversible serviceability limit state being exceeded is lower than that for a reversible serviceability limit state. A more onerous combination of actions is used for irreversible serviceability limit states than reversible serviceability limit states.

Key concept summary 3: Representative values of a variable action

There are four different representative values of a Variable Action. The characteristic value is a statistically extreme value. It is the main representative value, and the value generally defined in EN1991. The other representative values are called the combination value, frequent value and quasi-permanent value. They are determined by multiplying the *characteristic value by ψ_0, ψ_1 and ψ_2 respectively. The combination, frequent and quasi-permanent values are less statistically extreme than the characteristic value, so ψ_0, ψ_1 and ψ_2 are always less than 1.*

Key concept summary 4: Ultimate limit states

The Eurocodes explicitly establish six different ultimate limit states. Two of these, UPL and HYD, are specific to EN1997. Two are concerned with resistances: STR when verifying structural resistance and GEO when verifying the resistance of the ground. FAT is concerned with fatigue. EQU is principally concerned with ultimate limit states involving a loss of overall equilibrium. However, it has an important relationship with the single source principle (see key concept summary 5). Different partial factors on actions and geotechnical material properties are used for different ultimate limit states

Key concept summary 5: Single source principle

Application of the single source principle allows a single partial factor to be applied to the whole of an action arising from a single source. The value of the partial factor used depends on whether the resulting action effect is favourable or unfavourable. EN1990 allows the single source principle to be used for STR and GEO verifications. EQU addresses cases when minor variations in the magnitude or spatial distribution of a permanent action from a single source are significant.

Key concept summary 6: Combinations of actions

EN1990 establishes five different combinations of actions relevant to bridge design in UK. Different combinations of actions are used for verifying different limit states. They have different statistical likelihoods of occurring. The quasi-permanent combination is also used when analysing long-term effects. The differences between the combinations of actions concern: whether partial factors are applied; which representative values of variable actions are used; and, whether there is an accidental action included. The different combinations of actions are used in conjunction with the Eurocode 'material parts'. The Eurocode part generally states explicitly which combination is to be used in each SLS verification.

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DEVELOPMENT OF UK NA FOR EN 1990 ANNEX A2

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Abstract
Annex A2 was published as an amendment to BS EN 1990 in 2005 to provide the specific load combination rules, load combination factors (ψ_0,ψ_1,ψ_2) and the partial factors for permanent and variable actions (γ_G,γ_Q), that are applicable to bridges.
The common basis adopted for road, rail and footbridges, was to aim for an equivalent level of reliability to that previously obtained using the British Standards (Highways Agency Design Manual for Roads and Bridges BD 37 and BS5400-2 for example). Calibration studies were undertaken by Atkins for the Highways Agency and by Scott Wilson and Mott MacDonald for Network Rail and RSSB.
The outputs from these studies were used as the basis for selection of the design rules and the values of the load combination factors and partial factors, selected. The paper describes the work undertaken and provides information about the background to the development of the UK National Annex relating to BS EN1990 Annex A2.

Notation
ψ_0 Factor for combination value of a variable action
ψ_1 Factor for frequent value of a variable action
ψ_2 Factor for quasi-permanent value of a variable action
γ_G Partial factor for permanent actions
γ_Q Partial factor for variable actions

Introduction
Annex A2 was published as an amendment to BS EN 1990[1] in 2005 to provide the specific load combination rules, load combination factors (ψ_0,ψ_1,ψ_2) and the partial factors for permanent and variable actions (γ_G,γ_Q), that are applicable to bridges. It also contains requirements for serviceability limit state verification of bridges, including those relating to deformations and vibrations.

The common basis adopted for road, rail and footbridges, was to aim for an equivalent level of reliability to that previously obtained using the British Standards (Highways Agency Design Manual for Roads and Bridges BD37[9] (HA DMRB BD37) and BS5400-2[10] for example). Calibration studies were undertaken by Atkins for the Highways Agency and by Scott Wilson and Mott MacDonald for Network Rail and RSSB.

The outputs from these studies were used as the basis for selection of the design rules and the values of the load combination factors and partial factors, selected. The paper describes the work undertaken and provides the background to the development of the UK National Annex to support BS EN 1990[6], Annex A2.

The sections dealing with the design life and reliability, address the specific choices that have to be made in the design of bridges. The general concepts for design of bridges using the Eurocodes have been covered in paper 1-2/1 concerning the 'key concepts of EN 1990'.

Design Working Life for Bridges and Actions Beyond the Scope of EN1991

A fundamental design decision to be made when designing bridges to the Eurocodes (or other codes for that matter), is to establish the required working life. This will impact on the choice of partial factor values to achieve the required reliability of the structure commensurate with the required design life.

Actions that are not (currently) within the scope of the Eurocodes, may be established and combined in accordance with the procedures adopted in the Eurocodes, where it is possible to determine the level of reliability required for a specific project.

Design working life

Table 2.1 of BS EN 1990 contains 'indicative' values for the design working life of a bridge. The fact that the values are 'indicative', implies that alternative values may be appropriate and that Annex A2 of BS EN 1990 (**Clause A2.1.1(1), NOTE 3**) permits guidance on the application of **Table 2.1** for bridges to be provided in the National Annex.

BS EN 1990 defines the term 'design working life' as:

'assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.'

A value of 120 years has been selected to achieve consistency with the design practice of HA DMRB BD 37 and BS 5400-2. However, this value is appropriate for the majority of 'Persistent' design situations and alternative values may be appropriate for 'Transient' design situations, which are relevant for design for execution. **Table 3.1** of BS EN 1991-1-6[4] contains recommended values of the return period for determination of the characteristic values of 'climatic actions'. For 'Persistent' design situations for the design of permanent structures, the minimum return period should not be taken as less than 50 years, which is consistent with **Table 3.1** of BS EN 1991-1-6 for structures with a design life greater than 1 year.

Actions beyond the scope of EN 1991

The scope of EN 1991 is restricted to the coverage of the 'action' parts that are currently published. There are proposals by CEN to develop new parts of EN 1991 to address actions that are not currently covered ('waves and currents' and 'atmospheric ice loads' for example). However, it is recognised within BS EN 1990 that design for actions not covered by BS EN 1991, is sometimes necessary on a project-specific basis (ship collision on bridge piers for example). In this case, there is an expectancy that combinations of actions would be defined as permitted in BS EN 1990, **1.1(3)**.

For determination of actions beyond the scope of BS EN 1991, it is considered to be reasonable to use existing standards as a basis for their derivation (ISO and British Standards for example). In establishing the combination factors and partial load factors to be used with the actions determined in this way, it will be necessary to establish the design life and level of reliability required for the particular project concerned, taking account of the general probability that different action components may occur simultaneously. Provisions for defining actions caused by water and atmospheric icing, are included within BS EN 1991-1-6.

Combination Rules

Having established the individual actions that must be combined for design of bridges, it is necessary to know the rules for combination of actions when using the Eurocodes. The general principles and application rules for combination of actions and the combination rules for road bridges, footbridges and rail bridges, will be described.

General

Section **A2.2.1** of BS EN 1990 sets out the general principles and application rules for combination of actions.

The principles to be applied in considering combinations of actions are:

A2.2.1(6)P 'During execution the relevant design situations shall be taken into account'

This is quite explicit and simply means that the designer must consider what design situations are relevant during execution.

A2.2.1(7)P 'The relevant design situations shall be taken into account where a bridge is brought into use in stages'

In this case it is clear that relevant design situations must be considered for staged construction.

A2.2.1(9)P 'For any combination of variable traffic actions with other variable actions specified in other parts of EN 1991, any group of loads, as defined in EN 1991-2, shall be taken into account as one variable action'

Where traffic load components (vertical and longitudinal for example) are considered to act simultaneously, they may be considered as a 'load group'. The relevant 'load groups' for road bridges, footbridges and railway bridges are defined in BS EN 1991-2[5]. The principle here is that where a 'load group' is considered instead of the individual traffic load components, the 'load group' must be considered as a single variable action, which may be the 'leading' variable action or an 'accompanying' variable action depending on the case being considered.

The application rules cover the need to consider:

i. The simultaneous occurrence of actions where, for ‘physical or functional reasons’, they cannot act in combination.
ii. Combinations outside the scope of BS EN 1991 (see **A2.2.1(2), Note 1**).
iii. Use of expressions **6.9a** to **6.12b** for verification of ultimate limit states.
iv. Use of expressions **6.14a** to **6.16b** and **A2.4** for verification of serviceability limit states.
v. Where relevant, the simultaneous occurrence of traffic actions in accordance with BS EN 1991-2.
vi. The simultaneous occurrence of particular construction loads in combinations of actions.
vii. Wind actions and snow loads with loads due to construction activity.
viii. The simultaneous occurrence of thermal and water actions where relevant.
ix. The inclusion of prestressing actions in combinations of actions.
x. The effects of uneven settlements.

Unless limited by a ‘Principle’ or ‘Application’ rule in section **A2.2**, any combination of permanent and variable actions is, in theory, possible. An important limiting factor here is that actions need not be combined where it is impossible for them to occur simultaneously due to physical or functional reasons.

An example of this, and where there is a key difference between BS EN 1990 and BS 5400-2, is the requirement to consider wind and thermal actions in combination with traffic as the ‘leading’ action and vice versa. For design verification in accordance with the HA DMRB BD 37 and BS 5400-2, only a reduced live load is considered for wind (combination 2) or thermal (combination 3) combinations of actions.

Road bridges and footbridges

The combination rules for road bridges and footbridges have been established on the basis that BS EN 1990 Annex A2 (BS 5400-2) allows combination rules for actions to be defined in the National Annex or for the individual project, where these are necessary for geographical reasons.

The requirements for combination of actions defined in the UK National Annex to BS EN 1990, have been developed to achieve consistency with those in HA DMRB BD 37 (BS 5400-2). Generally, the combinations of actions are based on not more than two variable actions. Effects of actions that cannot exist simultaneously due to physical or functional reasons are not considered together in combinations of actions.

Based on the above principles, the UK National Annex to BS EN 1990, **NA.2.3.3.3**, **NA.2.3.3.4**, **NA.2.3.4.1** and **NA.2.3.4.2**, recommend that snow loads and combinations of wind and thermal actions, may generally be ignored in the UK for road bridges and footbridges, except for those with a roof where snow loads may need to be considered. Specific combinations of actions for footbridges on which pedestrian and cycle traffic is fully protected from all types of bad weather may be determined as appropriate for the individual project.

For combinations of actions including wind and temperature, the coincident occurrence of high winds together with extreme temperatures is highly unlikely. Therefore, combinations of wind and thermal actions are not likely to be significant for most road bridges. Furthermore, the nature of wind actions means that they are primarily concerned with horizontal effects on structural elements, whereas thermal actions generally result in vertical effects due to temperature differences through the structure depth, for beam and slab type structures. However, for frame-type structures, horizontal effects due to global expansion or contraction can be important. Therefore, for bridge structures that are highly sensitive to multi-directional action effects (compression chord of half-through footbridges for example), it may be necessary to consider combinations of wind and thermal actions.

Rail bridges

Specific combination rules for railway bridges, are set out in section **A2.2.4** of BS EN 1990.

Clause **A2.2.4(1)** permits combination rules for snow loads on railway bridges to be specified in the UK National Annex to BS EN 1990. The UK National Annex to BS EN 1990 confirms the requirement in **A2.2.4(1)** that snow loads do not in general need to be considered for the design of railway bridges. However, snow loads in combination with rail traffic, may be considered for individual projects, where there is a need, because there is a significant probability that such combinations of actions will occur (roofed railway bridges for example).

Clause **A2.2.4(2)** provides the rules for combination of traffic and wind actions. The second requirement states:

'vertical rail traffic actions excluding dynamic factor and lateral rail traffic actions from the "unloaded train" defined in EN 1991-2 (6.3.4) without wind forces for checking stability.'

The intention of this rule is to minimise the vertical stabilising forces and to maximise the horizontal destabilising forces for verification of stability (EQU) at the ultimate limit state. The clause should therefore include wind forces for checking stability and not exclude them as stated. This is an error and has been corrected in the April 2010 version of BS EN 1990 through a corrigendum.

Clause **A2.2.4(4)**, aims to set a maximum wind speed above which rail traffic is prevented from operating. The maximum wind speed that results in a wind pressure of 980Pa on the trafficked bridge is specified in the UK National Annex to BS EN 1991-1-4, **NA.2.45**. This wind speed is consistent with the maximum permitted wind load for trains to prevent, overturning in gales and, loss of electrical contact between the train and the overhead line (where provided), on exposed sections of the rail network.

Clauses **A2.2.4(5)** and **(6)** provide rules for combination of the aerodynamic effects of rail traffic in combination with wind actions. Clearly it is possible for trains to run when there is little or no wind and therefore it is logical for each action to be considered individually, as well as in combination, with each taken separately as the leading action. In addition, the presence of strong winds parallel to the direction of travel of the train, will enhance the resulting aerodynamic effects of the train nose passing pressure. Allowance is therefore required to be made for this in determination of the aerodynamic effects. Additionally, cross winds are also known to influence the magnitude of the train nose passing pressure. In the future it may be possible to take the effects of crosswinds into account with greater confidence when the results of current European research[11] are available and this may affect the wind loads that are permitted for trains to remain operational.

No restriction on the combination of wind and thermal actions is required for railway bridges. As for road bridges, combinations of wind and thermal actions are not likely to be significant for rail bridges. However, for bridge structures that are highly sensitive to multi-directional action effects, it is necessary to consider combinations of wind and thermal actions.

Combination Factors

Recommended values for the combination factors (ψ) for road and rail bridges and footbridges are set out in Tables **A2.1**, **A2.2** and **A2.3** of BS EN 1990, Annex A2.

Road bridges and footbridges

The design value of an action, or action combination, depends on the characteristic values of actions, the combination factors and the partial factors for actions. Hence the Nationally Determined Parameters (NDPs) for these in the UK National Annex to BS EN 1990, Annex A2, were calibrated together to broadly match the design actions derived using HA DMRB, BD 37 (BS 5400-2), for ULS and SLS verifications.

The combination factors for traffic and wind actions were established by assuming a reference period (time between uncorrelated load events) of 1 hour for traffic loading and 8 hours for wind loading. The 1 hour reference time for traffic is based on the duration of stationary traffic. Using the annual extreme value Gumbel distributions for time-dependent wind uncertainty and traffic uncertainty gives values of:

Ψ_0 traffic = 0.75 and Ψ_0 wind = 0.30.

Although this value for wind is lower than the value of 0.5 used in the National Annex, it was decided to use 0.5 to achieve consistency with the UK National Annex to BS EN 1990, Annex A1 (Buildings).

The combination rules for variable actions utilised by HA DMRB BD37 (BS 5400-2) and BS EN 1990 Annex A2, have very different formats. Because of this it is very difficult to achieve an exact match between HA DMRB BD 37 (BS 5400-2) and the UK National Annex to BS EN 1990. The two combination rules are compared in Figure 1 below, which shows the envelopes obtained by plotting the combination of the factored design values for wind and traffic actions, in accordance with HA DMRB BD 37 (BS 5400-2) and, the UK National Annex to BS EN 1990, normalised according to the respective maximum design values for actions. For each combination of wind and traffic actions in the equations (1) and (2) below (HA DMRB BD 37 (BS 5400-2), and for the UK National Annex to BS EN 1990) , the design combination value is divided by the full traffic load ($1.5Q_{Traff}$ for HA DMRB BD 37 (BS 5400-2) and $\gamma_{QTraff}Q_{Traff}$ for the UK National Annex to BS EN 1990) for the horizontal axis values, and by the full wind load ($1.4Q_{Wind}$ for HA DMRB BD 37 (BS 5400-2) and $\gamma_{QWind}Q_{Wind}$ for the UK National Annex to BS EN 1990) for the vertical axis values.

$$Q_{Design} = \max \left\{ \begin{array}{c} 1.5Q_{Traff} \\ 1.25Q_{Traff} + 1.1Q_{Wind} \\ 1.4Q_{Wind} \end{array} \right\} \quad (1)$$

$$Q_{Design} = \max \left\{ \begin{array}{c} \gamma_{QTraff}Q_{Traff} + \psi_{aff}\gamma_{QWind}Q_{Wind} \\ \psi_{Traff}\gamma_{QTraff}Q_{Traff} + \gamma_{QWind}Q_{Wind} \end{array} \right\} \quad (2)$$

In this way the plot provides a direct comparison of the differences obtained for traffic and wind action combinations using the two codes. It can be seen from Figure 1 that use of the UK National Annex to BS EN 1990 load combination factors will be consistently more conservative than HA DMRB, BD 37 (BS 5400-2). This comparison is purely in terms of the combination of variable actions and assumes that the HA DMRB, BD 37 (BS 5400-2) and the UK National Annex to BS EN 1990 design combination values for actions, are equal to the total design combined load.

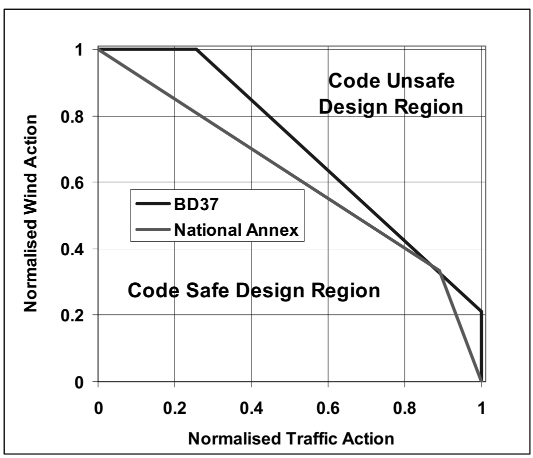

Figure 1. Comparison of variable action combination rules

The frequent and infrequent factors for LM1 traffic loading were derived using the following definitions:

$$\psi_{1freq} = \frac{Q_{1week}}{Q_{1000years}} \quad (3)$$

$$\psi_{1infq} = \frac{Q_{1year}}{Q_{1000years}} \quad (4)$$

Where Q_{1week}, Q_{1year} and $Q_{1000years}$ are traffic actions with 1 week, 1 year and 1000 year return periods respectively. The characteristic traffic action has a notional return period of 1000 years.

These values were derived from the probability distribution of the static component of the traffic actions. It should be noted that the actual nominal loads defined in the UK National Annex to BS EN 1991-2, have return periods which are considerably more than 1000 years for short spans. The frequent and infrequent combination factor values are therefore greater than the values based on the actual return period for short spans.

The explanation of the procedure adopted for derivation of the infrequent factor has been included for completeness. However, it should be noted that the decision was taken in the UK National Annex to BS EN 1990, Annex A2, **NA.2.3.6.3**, that the infrequent value of actions should not be considered.

Rail bridges

The values adopted for the combination factors (ψ) are the same as the recommended values in **Table A2.3** of EN 1990:2002 + A1:2005. These values were also included in ENV 1991-3[1] and UIC 776-1[12].

The combination factors were derived from the outputs of the UIC Panel of Structures Experts and summarised in their report[13]. This report, together with other background information, is identified in an RSSB report[14] which summarises the background to the railway traffic actions in BS EN 1991-2.

The ψ factors from **Table A2.3** of EN 1990:2002 + A1:2005 were used in the RSSB calibration studies for confirmation of the values for the partial factors[11,12].

Partial factors

The introduction of the Eurocodes represents a major change to the current UK practice for the design of bridges. For reasons of safety and economy, it is important to ensure that designs to the Eurocodes, along with their National Annexes, result broadly in the same level of overall safety and reliability as is implicit in the partial factors adopted in HA DMRB BD37 and BS 5400-2. However, the opportunity has also been taken to rationalise the reliability levels and to remove any inconsistencies or anomalies within the UK practice for different structure/material types.

Wind and thermal partial factors

The partial factors for wind and thermal actions on road bridges, footbridges and rail bridges in the UK National Annex to BS EN 1990, Tables **NA.A2.4(A)** and **NA.A2.4(B)**, have been adjusted to ensure a consistent level of reliability compared to BS 5400-2. This has involved derivation of the γ_Q factors taking account of the following:

i. Use of the γ_{fL} partial load factor in BS 5400-2.
ii. Return period factor (P_r) on wind velocity for 120 year 'design life' instead of the 50 year 'design life' implicit in the recommended values in BS EN 1990 Annex A2.

iii. Inclusion of the additional γ_{f3} factor used for determination of design load effects in BS 5400-2.

For example, the resulting partial factor for wind actions in BS EN 1990:2002, **Table NA.A2.4(B)** is given by

$$\gamma_Q = \gamma_{f3} \text{ x } P_r^2 \text{ x } \gamma_{fL} \qquad (5)$$

$$\gamma_Q = 1{,}1 \text{ x } 1{,}05^2 \text{ x } 1{,}4 = \underline{1{,}7}.$$

For thermal actions, the average of the return period adjustment factors for maximum shade air temperature (1,049) and minimum shade air temperature (1,139) has been used, and the resulting partial factor for thermal actions given in **Table NA.A2.4(B)** is given by,

$$\gamma_Q = \gamma_{f3} \text{ x } P_r \text{ x } \gamma_{fL} \qquad (6)$$

$$\gamma_Q = 1{,}1 \text{ x } 1{,}094 \text{ x } 1{,}3 = \underline{1{,}55}.$$

The values of γ_Q for wind and thermal actions in the UK National Annex to BS EN 1990, Table **NA.A2.4(C)**, have been established by selecting a value that is proportional to the reduction in the values for variable actions between Tables **NA.A2.4(A)** and **NA.A2.4(B)** and Table **NA.A2.4(C)**.

This is a pragmatic approach designed to achieve compatibility with existing practice. It should be noted that the wind map velocities in the NA to BS EN 1991-1-4 are based on a 10 minute mean whereas the values in the BS 5400-2 wind map are hourly mean values. However, this difference was examined in calibration studies and not found to result in any additional need to modify partial factors.

The UK National Annex to BS EN 1990, Tables **NA.A2.4(A)** to **NA.A2.4(C)**, permit adjustment of the γ_Q factors for persistent design situations where the duration of the design situation is taken into account directly (see BS EN 1991-1-4, **4.2(2)** and BS EN 1991-1-5, **A.2**). If this approach is followed, P_r is effectively determined for the particular case being examined and a reduced values of γ_Q equal to (1,1 x 1,4 ≈) 1,55 may be used for wind actions and (1,1 x 1,3 ≈) 1,45 may be used for thermal actions. The relevant notes in UK National Annex to BS EN 1990, Tables **NA.A2.4(A)** to **NA.A2.4(C)** explain that the return period for determination of the values of wind velocity and air shade temperature, may be adjusted for a mean return period equal to the design working life, but not less than 50 years.

For transient design situations, BS EN 1991-1-6, **3.1(5)**, recommends values of the return period for determination of characteristic values of climatic actions, in relation to the nominal duration of the design situation.

Road bridges and footbridges

A reliability based code calibration approach[17] was used in deriving the load partial factors for the UK National Annex to BS EN 1990, which comprised the following steps:

i. Selection of representative bridge types and design scenarios
ii. Establishment of the target reliability implicit in the previous UK design codes HA DMRB BD 37/BS 5400-2
iii. Calculation of the partial factors required to achieve the target reliability for each design configuration, and rationalisation into a common set
iv. Verification of the reliability implicit in the proposed partial factors.

The design configurations chosen for the calibration covered RC slab and steel-RC composite bridges of varying span lengths and carriageway widths. The loading scenarios included single and two-lane loading and calculation of critical moment and shear load effects. A number of bridge elements were notionally designed to the limit of BS 5400 and HA DMRB standards, covering the selected range of bridge types and load cases.

Probability distributions for the various loading and resistance parameters were established based on published research and previous work for the Highways Agency. The reliability for each of the notional designs was calculated to determine the reliability levels implicit in the previous UK design codes (BS 5400 and HA DMRB).

The average annual reliability index implicit in the previous UK design codes (BS 5400 and HA DMRB) for a 3m lane width and for loaded lengths greater than 20m, is 6.3 for bending and 5.3 for shear. The reliability levels are considerably higher for spans less than 20m. It is necessary that the partial factors for loads are the same for the different material types and limit states. Therefore an average target reliability index of 5.8 was adopted for the calibration of the partial factors for loads for use in the UK National Annex to BS EN 1990.

The partial factors for EQU (Set A), STR/GEO (Set B) and STR/GEO (Set C) verifications in the UK National Annex to BS EN 1990, were derived for traffic loading, concrete self-weight, steel self-weight and super-imposed dead loads (including the weight of surfacing). The reliability levels corresponding to the proposed partial factors in the UK National Annex to BS EN 1990, for combined permanent and traffic actions, were compared against the reliability implicit in BS 5400 and HA DMRB. The reliability levels for the UK National Annex to BS EN 1990 partial factors, show good agreement with the proposed target value of 5.8 for loaded lengths above 20m (see Figure 2). For loaded lengths below 20m, it can be seen that the reliability is higher than this, broadly in line with BS 5400 and HA DMRB reliability levels.

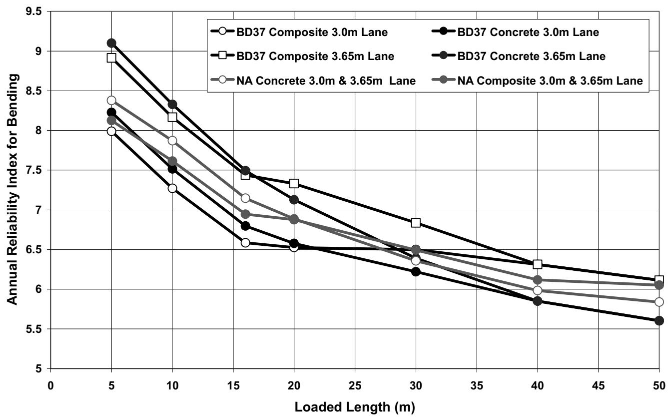

Figure 2. Results of reliability analysis for the UK National Annex to BS EN 1990 and HA DMRB BD 37 (BS 5400-2): Bending, Lanes 1+2

The partial factors for EQU (Set A), STR/GEO (Set B) and STR/GEO (Set C) verifications involving combinations of wind loading and for SLS limit states, were established using a similar calibration procedure.

Rail bridges

For rail bridges, the derivation of the partial factors in the UK National Annex to BS EN 1990, has been based on simple comparative calculations for typical UK railway bridges. These were undertaken on a deterministic basis, by studying the ratio between the design value of the load effect and the design resistance.

For rail traffic actions, different values of the partial load factor γ_Q for the three load model groups defined (LM71, SW/0 and HSLM; SW/2; Real trains, for example), have been established for EQU (Set A), STR/GEO (Set B) and STR/GEO (Set C) verifications.

Firstly, a value of $\gamma_Q = 1.45$ for LM71, SW/0 and HSLM, was found to provide a consistent margin of safety compared to design using BS 5400-2, in studies undertaken by RSSB and Network Rail[15],[16]. These studies confirmed the recommended value of γ_Q published in BS EN 1990:2002 + A1:2005, which was based on earlier studies undertaken by the UIC[13].

An additional value for $\gamma_Q = 1.40$ was established to provide adequate safety for heavy rail traffic, where greater control may be exercised on traffic operation. The UK National Annex to BS EN 1991-2 states that there is generally no requirement to design for SW/2 in the UK and other exceptional rail traffic (KIROW crane for example) although this value might apply where particular heavy rail traffic is being considered in design.

Further, a value for $\gamma_Q = 1.7$ was determined for individual real trains where the axle weight and speed combinations are beyond the range for vehicles covered by the usual load models

(LM71, SW/0 and HSLM). Network Rail can provide information on vehicles that fall into this category.

γ_Q values have been reduced approximately in proportion to the reduction in the values for variable actions in Tables **NA.2.4(A)** and **NA.2.4(B)** and Table **NA.2.4(C)**. Design Approach 1 (see BS EN 1997) is used where geotechnical actions or resistances are included for STR/GEO (Set C) verifications.

During execution of bridges constructed over particularly vulnerable obstacles, such as railways and motorways for example, it was recognised by the project team for development of the Eurocodes that a higher level of safety was required. Consequently, the partial factors for stability verifications (EQU) in such special cases were increased to 1.15 ($\gamma_{G,sup}$) and reduced to 0.85 ($\gamma_{G,inf}$) respectively.

Reliability of Bridges

The means of reliability differentiation for design of structures (including bridges) in accordance with the Eurocodes, is influenced by the selection of consequence classes (CC1, CC2 and CC3). The consequence classes (termed 'consequences classes' in BS EN 1990) are defined in terms of the consequences for loss of human life or economic, social or environmental impacts (Table B1, BS EN 1990). The criterion for classification of consequences is determined on the basis of an assessment of the importance of a structure or structural element. Classifications range from low (CC1), medium (CC2) to high (CC3). It is possible for individual elements of structures to have the same, a lower or higher consequence class than for the entire structure.

Most bridge failures would only result in medium consequences in the event of failure and the UK National Annex to BS EN 1990, **NA.3.2.1** Annex B, specifies that bridges should normally be treated as consequence class 2, although the actual consequence call is established on a project specific basis and may be subject to client requirements. The recommendation of consequence class 2 is based on the likelihood of traffic or pedestrians being on or below a typical bridge at the instant of failure. For lightly trafficked road or rail bridges (on a rural network for example) this could be too conservative and a lower consequence class could be appropriate. However, for heavily trafficked bridges, bridges at busy intersections of key routes or for large span bridges over estuaries, for example, consequence class 3 might well be appropriate.

BS EN 1990 Annex B, suggests that different reliability classes may be associated with different consequence classes and that for different consequence class structures a different target level of reliability index, β, may be appropriate. It continues by suggesting that partial factors might be adjusted depending upon the consequence class. The calibration of the partial factors included in the UK National Annex to BS EN 1990 has been undertaken to provide an acceptable level of safety across all bridge types and it is therefore recommended that no such adjustments to partial factors are made, unless justified by the circumstances of a specific project.

Conclusions

The UK National Annex to BS EN 1990 has been developed to achieve a consistent level of reliability compared to that obtained using HA DMRB BD 37 and BS 5400-2. The values for the partial factors for permanent actions (γ_G) and for variable actions (γ_Q), for persistent design situations, have been selected to provide equivalence with previous design practice and are consistent with Bridges designed using the UK National Annex to BS EN 1990 having a design working life of 120 years.

The combination rules (section **A2.2.1** of BS EN 1990) require the combination of any actions so long as they can exist simultaneously due to physical or functional reasons and they are within the scope of EN 1991. Certain combinations of actions are excluded by the specific combination rules for road bridges, footbridges and rail bridges in **NA.2.3.3**, **NA.2.3.4** and **NA.2.3.5**, where they are not considered to act simultaneously to any significant degree.

Tables **NA.A2.4(A)** to **NA.A2.4(C)** of the UK National Annex to BS EN 1990 include partial factors for wind and thermal actions that account directly for the difference between a design working life of 120 years and the 50 year return period used to establish the characteristic values of such actions. However lower γ_Q factors are also provided for wind and thermal actions in persistent design situations that may be used where the duration of the design situation is taken into account directly. The return period for determination of the values of wind velocity and air shade temperature, may be adjusted for a mean return period equal to the design working life, but not less than 50 years. For transient design situations, BS EN 1991-1-6, **3.1(5)**, gives recommended return periods for the determination of values of climatic actions, related to the nominal duration of the design situation.

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Design Performance Requirements for Railway Bridges in BS EN 1990:2002 Annex A2

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Abstract

BS EN 1990 Annex A2 contains the design performance requirements that are applicable to railway bridges. Compliance with the requirements limiting deformations and deck accelerations is intended to ensure the safe operation of the railway and comfort of passengers.

While limiting deformations is required for railway bridge design to BS 5400-2 and associated railway group standards, checking the bridge behaviour for the dynamic effects of railway traffic has historically been limited to applying a dynamic factor Φ to the static load obtained from the design load models (RU and SW/0) in BS 5400-2. However, where bridge decks fall outside of the limits assumed in determining Φ, a dynamic analysis is required to ensure that the deformation and vibration limits set in Annex A2 of BS EN 1990 are met.

The background to the differences between the design performance requirements adopted for designs to BS 5400-2 and the requirements of BS EN 1990 Annex 2 and BS EN 1991-2, will be examined. The emerging requirements for clarification and amendment of the design performance requirements in Annex A2 of BS EN 1990 will also be highlighted.

Notation

Φ	dynamic factor
α	load classification factor
L	effective span length
δ	deformation

Introduction

Rail bridges are designed to ensure the safe passage of the railway traffic. This is achieved by ensuring that the stiffness of the bridge is sufficient to prevent excessive deformations that could lead to overstress in the rails, dangerous twisting of the tracks leading to derailment or an excessive dynamic response of the bridge leading to premature failure. The bridge must also not cause uncomfortable vibrations for passengers. BS EN 1990 Annex A2[1] specifies the performance criteria for railway bridges to ensure these levels of safety and comfort are met.

The limits for allowable deformations specified in BS EN 1990 Annex A2, generally align with the limits specified in UIC 776-3R[2], although the application of the load classification factor, α, (BS EN 1991-2[3], 6.3.2(3)P), means that the BS EN 1990 Annex A2 railway loads are typically greater than those defined in UIC 776-3R as amended by GC/RC5510[4] (now withdrawn).

Dynamic effects on railway bridges were generally included in the quasi-static analysis required in BS 5400-2[5] and UIC 776-3R. This design approach means that the static classified vertical load effects are factored to account for the dynamic effects of the loading from trains, through the application of a dynamic factor Φ. However, this approach is valid only for bridges that satisfy certain stiffness criteria, and for standard railway traffic travelling at up to 200km/h. Where bridges or bridge floors are outside of the stiffness criteria, or line speeds exceed 200 km/h, a bridge specific dynamic analysis is required in accordance with the UK National Annex to BS EN 1991-2[6], **NA.2.50**.

The following sections compare the BS 5400-2 requirements and methodologies with those of BS EN 1990 Annex A2.

Design Performance – Safety: Deformation

In addition to the structural capacity of the bridges, the bridge is designed to ensure that the track is not overstressed or that trains are subjected to sudden changes in the track profile through excessive deformation. BS EN 1990 Annex A2 specifies deformation limits to ensure that bridge performance will not contribute to an unsafe railway. The limits take into account the mitigating effects of track maintenance that overcome, for example, the effects of settlement and creep. The performance requirements are described in the following sections and comparisons are made with the historical requirements.

Track twist limitations

The twist of the bridge, measured along the centreline of each track, is limited to minimise the risk of derailment (see Figure 1). The track twist must be checked on the approach to the bridge, across the bridge and on departure from the bridge, and also to include the effects of adjacent tracks being loaded, where the bridge supports more than one track.

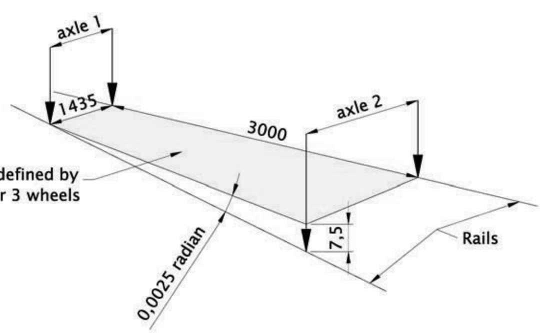

Figure 1. BS EN 1990 Annex A2 definition of and limit for total track twist

BS EN 1990 Annex A2, **A2.4.4.2.2 (2),** requires the track twist to be checked due to the action of railway traffic loads only and BS EN 1990 Annex A2, **A2.4.4.2.2 (3**) requires the total track twist to be checked, i.e. with the track profile considered in addition to the deformation of the bridge under the railway traffic loads.

BS EN 1990 Annex A2, **A2.4.4.2.2 (2)**, permits the National Annex to specify the allowable twist limit when checking the twist under railway traffic load only and the UK National Annex to BS EN 1990 recommends that the requirements are specified for the individual project. Network Rail's design standard, NR/L2/CIV/020[7] (planned for publication Spring 2011), states that the requirements of **A2.4.4.2.2 (2)** to check the maximum twist shall not apply and that the only total twist shall be considered. This is because the requirements of BS EN 1990 Annex A2, **A2.4.4.2.2 (2)** are considered to be conservative as a traffic safety limit and UK experience has demonstrated that compliance with the total twist limits is acceptable.

The total twist limits of BS EN 1990 Annex A2, **A2.4.4.2.2 (3)** are the same as the limits in UIC 776-3R. For a track gauge of 1,435m and two axles spaced 3m apart, no wheel shall be more than 7,5mm out of the plane defined by the other three wheels (see Figure 1). This 7,5mm/3m total twist limit is the equivalent of the 0,0025 radians limit defined in UIC 776-3R.

However, the applied load for checking the total twist is different. UIC 776-3R, as amended by GC/RC5510 (now withdrawn), requires the dynamic factor Φ to be applied to the specified design loading (i.e. vertical railway traffic loads and centrifugal effects) with partial factors equal to 1.0.

BS EN 1990 Annex A2, **A2.4.4.2.2(1)P** states that the loads to apply are, the vertical and centrifugal effects of LM71 (also SW/0 and SW/2 as appropriate) multiplied by the dynamic factor Φ and the load classification factor α. This results in an applied railway traffic load higher than that for designs using UIC 776-3R as the UK National Annex to BS EN 1991-2, **NA.2.48** specifies an α value of 1,10 and NR/L2/CIV/020 specifies an α value of 1,21. The latter includes a robustness factor (γ_{det} = 1.1) to satisfy the High Speed Technical Specification for Interoperability requirements to maintain safety and reliability over the life of the structure and shall be applied to all designs for new and, where possible, replacement bridge designs.

The appropriate value for the dynamic factor Φ has always been subject to interpretation as bridge decks deform both transversely and longitudinally and a different dynamic factor can be applicable where the stiffness of the transverse and longitudinal elements is significantly different and for decks with a significant skew. BS EN 1990 Annex A2 does not provide additional guidance on the appropriate value of Φ to use, but non contradictory complimentary information is included in NR/L2/CIV/020, allowing a composite dynamic factor be calculated to represent the variable influence of the longitudinal and transverse elements.

Vertical deformation limitations

The vertical deformation limits specified in BS EN 1990:2002 Annex A2, **A2.4.4.2.3** for the maximum total vertical deformation and rotations at the bridge ends ensure acceptable vertical track radii and robust structures (see Figure 2a). The deck end rotation limits ensure that additional rail stresses and uplift forces on rail fasteners are minimised, along with minimising angular discontinuities at switches or rail expansion devices near the bridge.

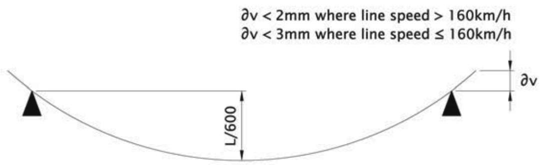

Figure 2a. BS EN 1990:2002 Annex A2 vertical deformation limits

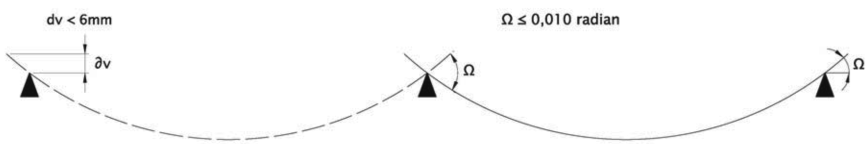

Figure 2b. UIC776-3R vertical deformation limits

The total vertical deformation limit in BS EN 1990 Annex A2, **A2.4.4.2.3(1)** is L/600, measured along any track, where L is the effective span. A comparable safety limit is not specified in UIC 776-3R: UIC 776-3R, **section 4** only states that the total vertical deflection must not encroach into the required headroom. UIC 776-3R, **section 6** recommends that the rate of change of angle at the simply supported ends of the bridge deck is checked against the limits in UIC 776-3R, **table 1**. The change of angle is limited to 0,010 radian where ballasted track is provided on both sides of a joint at the end of the bridge, and 0,005 radian where direct fastened track is provided on oneside of the deck end joint and ballasted track provided on the other side of the joint (see Figure 2b).

BS EN 1990 Annex A2, **A2.4.4.2.3** refers to BS EN 1991-2, **6.5.4** for the rotation limitations at the bridge ends of ballasted track. BS EN 1991-2, **6.5.4.5.5(3)P** limits the vertical deformation to 3 mm for line speeds up to 160 km/h and 2 mm for line speeds over 160 km/h. For a typical bridge overhang past the bearings of 300mm, the 3mm limit (up to 160 km/h) in BS EN 1990 Annex A2 will be less than the rotation allowed in UIC 776-3R **Table 1**. Rotation limits for non ballasted track are required to be specified by the individual project in the UK National Annex to BS EN 1990, **NA.2.3.11.6**.

As with the track twist checks, the applied load for checking vertical deformation appears to be different. UIC 776-3R, as amended by GC/RC5510 (now withdrawn) requires the dynamic factor Φ to be applied to the specified design loading (i.e γ_{fL} x RU or SW/0). BS EN 1990:2002 Annex A2, **A2.4.4.2.3(1)** specifies the classified characteristic vertical load be applied, i.e. LM71 (SW/0) multiplied by α, but no mention is made of the requirement to apply the dynamic factor (Φ). This is inconsistent with UIC 776-3 and the checks for twist of the bridge deck (BS EN 1990 Annex A2, **A2.4.4.2.2**) and transverse deformation (BS EN 1990:2002 Annex A2, A2.4.4.2.3), where the need to apply the dynamic factor is specifically stated. This oversight will be corrected in a future amendment of BS EN 1990 Annex A2 and **A2.4.4.2.3(1)**, which will require the dynamic factor (Φ) to be applied when checking the vertical deformation of a deck.

Transverse deformation limitations

Transverse deformation of the bridge is required to be limited to ensure that the horizontal track alignment remains acceptable (see Figure 3).

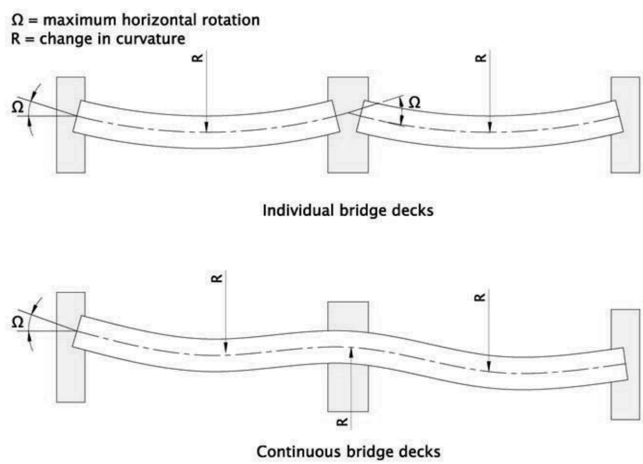

Figure 3. Definition of transverse deformations

The need for transverse deformation limits is generally more likely to feature where the transverse stiffness of the deck is much less than the longitudinal stiffness, although it may also be an issue for skew decks.

BS EN 1990 Annex A2, **A2.4.4.2.4** specifies the maximum change of radius of curvature and maximum horizontal rotation in **Table A2.8**. The limits depend on the number of consecutive decks and the line speed. The limits are the identical to those in UIC 776-3R.

The loads to be applied in accordance with BS EN 1990 Annex A2 are LM71 (SW/0) multiplied by Φ and α, including wind, nosing, centrifugal and thermal effects. Although not categorically stated in UIC 776-3R, it is implied that the same loads are to be applied as those in BS EN 1990, Annex A2, with the exception that α is not required to be applied to the railway traffic loads for checking the UIC 776-3R deformation limits.

Longitudinal deformation limitations

The longitudinal displacement of the end of the upper surface of the deck due to longitudinal displacement and rotation of the bridge deck end shall be limited to minimise rail stresses and to minimise disturbance to the track ballast and the adjacent track formation (see Figure 4). The check of longitudinal deformation is a new concept introduced in BS EN 1990, Annex A2, **A2.4.4.2.5**. The requirements for determination of the required combined response of the structure and track, are defined in BS EN 1991-2, **6.5.4.5.2.**

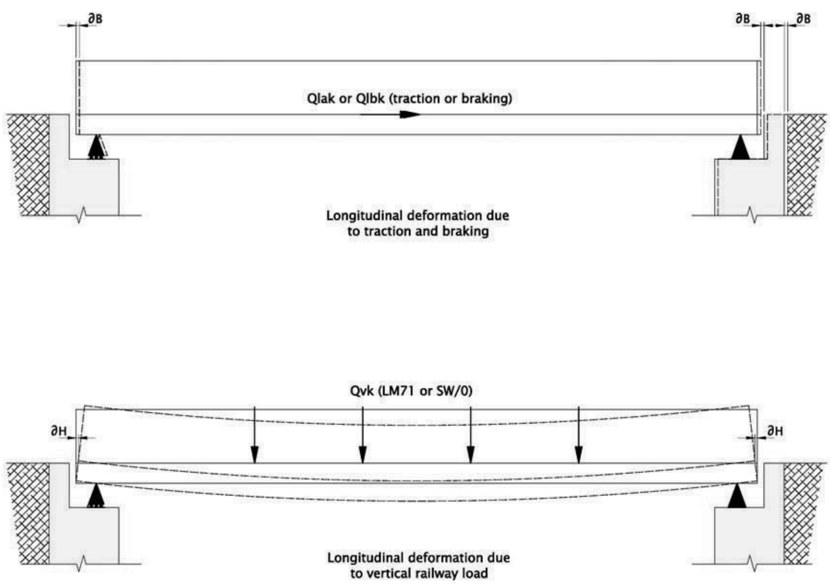

Figure 4. Definition of longitudinal deformations

BS EN 1991-2, **6.5.4.5.2(1)P** requires that the deformation, at the bridge end between two adjacent decks, or between the deck and the substructure, under traction and braking, is checked. The loads to be applied when checking deformations are the larger of the longitudinal traction and braking loads derived in accordance with the UK National Annex to BS EN 1991-2, **NA.2.45.2** or **BS EN 1991-2, 6.5.3.** The longitudinal deformation limits depend on the track details and are typically 5 mm for continuous welded rails without rail expansion devices or with a rail expansion device at one end of the deck, or 30 mm for rail expansion devices at both ends of the deck.

In addition to checking the longitudinal deformation under longitudinal loads, the longitudinal deformation of the bridge ends under the vertical loads also needs to be considered. BS EN 1991-2, **6.5.4.5.2(2)P** specifies that up to two tracks may be loaded with classified vertical traffic loads (i.e. LM71 (SW/0) multiplied by α) with any associated dynamic effects neglected. The longitudinal deformation limits depend on whether the combined response of the track and structure has been considered, being 8 mm when the combined behaviour of the structure and the track is taken into account, or 10 mm when the combined behaviour of the structure and the track is neglected.

Design Performance – Safety: Vibration

The design performance of railway bridges has historically been assured through the application of a dynamic factor Φ to the static load obtained from the design load models (RU and SW/0) in BS 5400-2. For the traffic that generally uses mainline railway bridges in the UK, a mix of passenger trains with a maximum speed of 200 km/h, and freight trains with a maximum speed of 120 km/h, has been assumed to comply with the BS 5400-2 load models. The application of a dynamic factor Φ to the loads calculated in accordance with the requirements of BS 5400-2, has in practice been deemed to provide an adequate level of safety for the dynamic effects in most cases. The dynamic performance of a railway bridge is checked indirectly by complying with, the live load deflection limits, and limits for the first natural frequency of bending (n_0), set out in UIC 776-3R. These limits are the same as those included in the UK National Annex to BS EN 1991-2, **Figure NA.12**.

However, where train speeds exceed the maximum values stated above, or where resonance can occur, the dynamic factor calculated from BS 5400-2 may not be adequate and there could be a risk of overloading a bridge or of premature fatigue failure. To address this situation, the UK National Annex to BS EN 1991-2 contains a procedure to establish whether a bridge-specific dynamic analysis is necessary.

Where a dynamic analysis is required, the deformation and vibration limits set out in BS EN 1990 Annex A2, are intended to ensure the safe performance of railway bridges subject to passenger trains travelling at speeds greater than 200 km/h. The High Speed Load Model (HSLM) was introduced to replicate the dynamic characteristics of real high speed trains for this purpose. The design requirements within BS EN 1990 Annex A2 were developed for typical Continental European slab-type railway bridge decks. However, there are railway bridge types that are prevalent in the UK (half-through types for example), which would not comply with the BS EN 1990 Annex A2 limits at speeds lower than 200 km/h and the UK National Annex to BS EN 1991-2, **NA.2.50**, requires consideration of these railway bridge types and for speeds below 200 km/h also.

The need for dynamic analysis

The UK National Annex to BS EN 1991-2, **NA.2.50** provides the flow charts to determine whether a dynamic analysis is required. In addition to the quasi-static analysis, the design may be checked to ensure resonance does not occur (as this could lead to unacceptable deformations and a reduced fatigue life), and that the deck response (acceleration) is within the limits set out in BS EN 1990 Annex A2. The UK National Annex to BS EN 1991-2, **Figure NA.12** is applicable for simple structures (i.e. those structures that exhibit only longitudinal line beam behaviour) and the UK National Annex to BS EN 1991-2, **Figure NA.13** is applicable for both simple and complex structures (i.e. those structures that require deck/floor elements to distribute axle/wheel loads to primary longitudinal elements).

The UK National Annex to BS EN 1991-2, **NA.2.50** allows all choices and options relating to dynamic analysis to be specified for the individual project. Consequently, where a dynamic analysis is necessary, reference to NR/L2/CIV/020 is required. Much of the information contained in NR/L2/CIV/020 was established following research undertaken for the West Coast Mainline Upgrade project. Details of the background research supporting the requirements and limits stated in NR/L2/CIV/020, for railway bridge types common on the UK railway network, are described in the paper *Permissible Deck Accelerations for Rail Bridge Dynamic Assessments,* Norris. P et al.[8].

Dynamic analysis design rules

Where a dynamic analysis is required, the designer is required to check the effect of real trains (axle loads and spacings to be specified for the individual project) and, where the route is part of the high speed Trans European Network (TENs route), the load effects attributable to the High Speed Load Models (HSLMs). Two models, HSLM A and HSLM B, are defined in BS EN 1991-2, **6.4.6.1.1(4)** and **(5)** respectively. Each HSLM represents a Universal Train with variable coach lengths. The pair of HSLMs together represent the dynamic load effects of articulated, conventional and regular high speed passenger trains, which comply with the requirements for the European Technical Specification for Interoperability on high speed routes (HS INS TSI).

BS EN 1991-2, **6.4.6.1.1 (6), Table 6.4**, specifies requirements for the application of HSLM A and HSLM B for analysis, in terms of particular structure types and for particular span ranges. It is permitted to specify additional requirements for the application of HSLM A and HSLM B in the National Annex. The UK National Annex to BS EN 1991-2, **NA.2.54** allows the individual project to specify additional requirements for the application of HSLM but these are to be specified for the individual project.

Bridge deck acceleration limitations

BS EN 1990, Annex A2, **A2.4.4.2.1(4)**, provides limits for bridge deck acceleration associated with particular loading frequencies. The recommended deck acceleration limits are:

i. $\gamma_{bt} = 3{,}5\ m/s^2$ for ballasted track.
ii. $\gamma_{df} = 5{,}0\ m/s^2$ for directly fastened track.

These acceleration limits are associated with loading frequencies up to the greater of:

i. 30 Hz.
ii. 1,5 times the frequency of the fundamental mode of vibration of the member being considered.
iii. The frequency of the third mode of vibration of the member.

The UK National Annex to BS EN 1990, **NA.2.3.11.2**, advises that the maximum peak values of bridge deck acceleration and the associated frequency limits, should be determined for the individual project and Railway Group Standard, GC/RT5112[9], **3.3.2**, states that the recommended peak values for deck acceleration shall be used. The GC/RT5112 recommendations are valid only for typical Continental European slab-type railway bridge decks. Shake table testing in Germany, commissioned by Network Rail as part of the West Coast Route modernisation project, has demonstrated that the BS EN 1990, Annex A2, **A2.4.4.2.1(4)** limits are onerous for typical UK railway half through bridge types. Therefore the requirements in NR/L2/CIV/020 include increasing the limit $\gamma_{bt} = 5{,}0\ m/s^2$ for ballasted track for short half wave lengths (i.e. short enough to not disturb more than two sleepers).

GC/RT5112 will be amended to clarify the limitations of the recommended values in EN 1990, Annex A2, **A2.4.4.2.1(4)**, at the earliest opportunity.

Design Performance – Safety: Other Requirements

In addition to the performance requirements described in detail in BS EN 1990:2002 Annex A2 and standards referred to therein, BS EN 1990:2002 Annex A2, **A2.4.4.1(2)P** specifies that unrestrained uplift at bearings shall be avoided to prevent premature bearing failure.

Design Performance – Passenger Comfort

Passenger comfort depends on the vertical acceleration experienced inside the coach. Although subjective, research by the Office for Research and Experiments (ORE) has derived a number of comfort levels; 'acceptable', 'good' and 'very good', with typical levels of acceleration associated with each. The bridge response, train suspension response and condition of the track, all influence the vertical acceleration experienced by the passengers. A simplified approach with rules based on checking the vertical deflections of a railway bridge, has been established and although not specifically stated, it is assumed the approach is valid for railway bridges with a natural frequency within the limits in BS EN 1991-2, **Figure 6.10**.

Deformation limitations

BS EN 1990, Annex A2, **Figure A2.3**, provides charts of the span / deflection limits plotted against span for three or more simply supported spans and for a 'very good' comfort level (see Figure 5). The limits may be factored where appropriate in accordance with **A2.4.4.3.2 (4), (5) and (6)** for other comfort levels, span numbers and span arrangements. The UK National Annex to BS EN 1990, Annex A2, **NA.2.3.11.10**, allows the individual project to specify the level of comfort required. NR/L2/CIV/020 requires a 'good' level of comfort for all bridges except for bridges on a primary route, or where the line speed exceeds 145km/h, where a 'very good' level is specified.

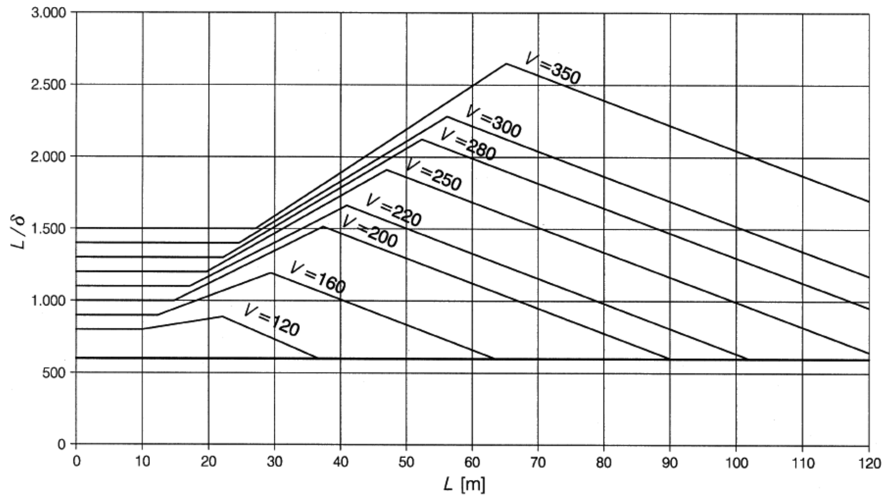

Figure 5: Vertical deformation limits for very good passenger comfort and a 3 span simply supported bridge. (BS EN 1990:2002 Annex A2, Figure A2.3)

This approach is similar to that in UIC 776-3R which gives a number of tables with span / deflection limits for selected bridge arrangements. The values in UIC 776-3R do not cover such a range of speeds as in BS EN 1990, Annex A2. A comparison of the two indicates that the BS EN 1990, Annex A2 limits generally allow a greater deformation than UIC 776-3R.

As with the safety performance requirements, the load applied in accordance with BS EN 1990, Annex A2, is LM71 multiplied by the dynamic factor Φ and the load classification factor α. For passenger comfort, BS EN 1990, Annex A2, **A2.4.4.3.1(2)** specifies a value of $\alpha = 1{,}0$. This is the same as the load specified in UIC 776-3R as amended by GC/RC5510 (now withdrawn).

It is worth noting that the passenger comfort span / deflection limits are usually more stringent than the L/600 vertical deformation safety limit in BS EN 1990, Annex A2, **A2.4.4.2.3(1)**.

Design Performance – Future Amendments

The European Railway Agency, has submitted a proposal to CEN for amendment of BS EN 1990 Annex A2. The proposals are for revision of section **A2.4.4** to:

i. clarify the purpose of the verifications regarding deformations and vibrations for railway bridges,
ii. review and revise the recommended limiting values for deformations and vibrations for railway bridges, especially limiting values for maximum acceleration of a bridge deck, longitudinal displacements at the ends of a bridge deck and limits relating to the transverse vibration of a bridge deck.

For the purpose of ensuring compatibility with the European Technical Specifications for Interoperability (TSIs), it is necessary to improve understanding of the purpose of the performance limits set, in particular the distinction between safety and other performance requirements. It is recognised that further work undertaken by the UIC since the initial development of BS EN 1990, Annex A2, has identified areas for further study covering:

i. potential conservatism of the acceleration and deformation limits,
ii. distinction between safety and comfort limits for track twist,
iii. clarification of the purpose of the limits set i.e. safety or other
iv. compatibility with interoperability limits for axle load/speed

Conclusions

BS EN 1990 Annex A2 provides requirements for the design performance of railway bridges, through limiting deformations and deck accelerations, to ensure the safe operation of the railway and the comfort of passengers. The deformation checks will be familiar to the railway bridge designer who will not notice significant differences compared with the existing requirements of UIC 776-3R as amended by GC/RC5510 (now withdrawn). However, BS EN 1990 Annex A2 does introduce a number of checks not previously routinely checked in design. These include checking of longitudinal deformations and, more significantly, checking the response of the structure to high speed railway traffic to ensure resonance or enhanced dynamic effects are avoided.

The opportunity has also been taken in this paper to identify areas where additional clarification is required, such as:

i. $\alpha = 1{,}21$ (specified in NR/L2/CIV/020) shall be used in the deformation checks as appropriate.
ii. In checking the vertical deformation of the bridge against the L/600 limit in BS EN 1990 Annex A2, A2.4.4.2.3, the dynamic factor Φ shall be applied.

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SESSION 1-3:

EN 1991 – ACTIONS AND EN 1998 – SEISMIC DESIGN

Bridge Design to Eurocodes: UK Implementation
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OVERVIEW OF ACTIONS IN EN 1991 AND EN 1998 FOR BRIDGE DESIGN

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Abstract

The constituent parts of BS EN 1991 (Eurocode 1) comprise:

BS EN 1991-1-1 Densities, self-weight and imposed loads
BS EN 1991-1-2 Actions on structures exposed to fire
BS EN 1991-1-3 Snow Loads
BS EN 1991-1-4 Wind actions
BS EN 1991-1-5 Thermal actions
BS EN 1991-1-6 Actions during execution
BS EN 1991-1-7 Accidental actions
BS EN 1991-2 Traffic loads on bridges
BS EN 1991-3 Actions induced by cranes and machinery
BS EN 1991-4 Actions in silos and tanks

The coverage of this conference is confined to those parts of BS EN 1991 that are relevant to bridges. These parts are highlighted bold in the above list. Separate papers describing the background to the UK National Annex (NA) and, where relevant, the associated Published Documents (PDs), are included for BS EN 1991-1-4, BS EN 1991-1-5, BS EN 1991-1-7 and BS EN 1991-2. The coverage of this paper is primarily concerned with differences in the treatment of actions (loads) in BS EN 1991 compared to previous design practice (BS 5400-2 and HA DMRB BD 37 for example).

Also covered is a brief overview of the way in which seismic actions are derived using BS EN 1998 for UK bridges. Since seismic actions are not covered specifically in BS 5400-2 and HA DMRB BD 37, these requirements cover new ground. More detailed discussion of earthquake design guidance for UK bridges is included in a separate paper (paper 1-3/6, Booth *et al* 2010).

This paper will highlight the issues that will need to be addressed in future in the light of experience gained from implementation of the Eurocodes.

Notation

Gk *characteristic value of a permanent action (BS EN 1990);*
$G_{k,sup}$ *upper characteristic value of a permanent action (BS EN 1990);*
$G_{k,inf}$ *lower characteristic value of a permanent action (BS EN 1990);*

γ_{fL}	*partial load factor (BS 5400-2 and HA DMRB BD 37);*
I	*importance factor (BS EN 1998);*
q	*behaviour factor (BS EN 1998);*

Introduction

BS EN 1991 (Eurocode 1) comprises ten parts that cover the range of permanent and variable actions that are applied to structures within the scope of the Eurocodes. These are:

BS EN 1991-1-1 Densities, self-weight and imposed loads[2]
BS EN 1991-1-2 Actions on structures exposed to fire
BS EN 1991-1-3 Snow Loads[3]
BS EN 1991-1-4 Wind actions[4]
BS EN 1991-1-5 Thermal actions[5]
BS EN 1991-1-6 Actions during execution[6]
BS EN 1991-1-7 Accidental actions[7]
BS EN 1991-2 Traffic loads on bridges[8]
BS EN 1991-3 Actions induced by cranes and machinery
BS EN 1991-4 Actions in silos and tanks

The coverage of this conference is confined to those parts of BS EN 1991 that are relevant to Bridges. These parts are highlighted bold in the above list. Separate papers describing the background to the UK National Annex and, where relevant, the associated Published Documents (PDs), are included for BS EN 1991-1-4, BS EN 1991-1-5, BS EN 1991-1-7 and BS EN 1991-2. The coverage of this paper is primarily concerned with differences in the treatment of actions (loads) in BS EN 1991 compared to previous design practice (BS 5400-2 and HA DMRB BD 37 and HA DMRB BD 37, for example).

Also covered is a brief overview of the way in which seismic actions are derived using BS EN 1998-1[10] and BS EN 1998-2[11] for UK bridges. Since seismic actions are not covered specifically in BS 5400-2[18] and HA DMRB BD 37[19], these requirements cover new ground. More detailed discussion of earthquake design guidance for UK bridges is included in a separate paper (paper 1-3/6, Booth et al 2010[22]) which summarises the requirements of BS EN1998-2 for the design of bridges in areas of very low seismicity (such as the UK) and the supplementary advice and complementary guidance specific to the UK given in (respectively) the UK National Annex to BS EN1998-2 and in the BSI Published Document PD 6698[17].

The paper will highlight the issues that will need to be addressed in future in the light of experience gained from implementation of the Eurocodes.

'Actions' Not 'Loads'

Like it or not, we are all going to have to get used to the term *'actions'* instead of *'loads'*, although both are still used in the Eurocodes. This is potentially confusing and it is perhaps helpful to examine the use of relevant terminology, the use of the alternative terminology and, to attempt to provide some explanation.

Based on the experience of those involved in the development of the Eurocodes, an important reason for the adoption of particular terminology, is the need to ensure consistent language that is readily translated into other European languages, particularly French and German. With this in mind, a useful starting point is to consider the existing definitions in BS 5400-2 and HA DMRB BD 37, and BS EN 1990[1].

In BS 5400-2 and HA DMRB BD 37, the term *'loads'* is defined as:

'external forces applied to the structure and imposed deformations such as those caused by restraint of movement due to changes in temperature'

In BS EN 1990, the term *'action'* is defined as:

a) *Set of forces (loads) applied to the structure (direct action);*
b) *Set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).*

It may be inferred from these definitions that the terms *'load'* and *'action'* may be taken to mean the same thing i.e. they are external *'forces'* or *'loads'* applied to the structure and both are also taken to include imposed deformations due to restraint of movement (temperature and accelerations due to earthquake for example).

However, the Concise Oxford Dictionary[29] provides the following definitions for *'load'*:
'a unit of measure or weight of certain substances';
'the weight or force borne by the supporting part of a structure'.

Similarly, a *'force'* is defined as:
'an influence tending to cause the motion of a body';

and the:
'intensity of this equal to the mass of the body and its acceleration' (Newton's 2nd Law)

The Eurocodes therefore use the term *'load'* to mean a force applied to a structure and *'actions'* is a generic term taken to mean an imposed *'load'* or imposed *'deformation'*. This is consistent with the dictionary definitions and illustrates the point that, although well established and accepted, the use of the term *'load'* in BS 5400-2 and HA DMRB BD 37 to mean imposed forces and imposed deformations, is not strictly correct.

For this paper, the Eurocode terminology will be used throughout (i.e. *'actions'* meaning an imposed load or deformation and *'load'* meaning a force applied to a structure). *'Actions'* and *'loads'* may be classified as permanent, variable, accidental, seismic or geotechnical. Accidental and geotechnical *'actions'* and 'loads' will not be considered in any detail within this paper.

Coverage of BS EN1991, BS EN1998 and BS 5400-2 and HA DMRB BD 37

Action coverage

The ‘actions’ due to permanent and transient loads in BS 5400-2 and HA DMRB BD 37, are covered in various parts of BS EN 1991 and BS EN 1998. Only those ‘actions’ (loads) which are not covered in other papers (i.e. self-weight, construction, shrinkage and creep, and snow), are considered in this paper. ‘Actions’ arising from earth pressure covered in BS EN 1997-1[9] are also not considered further in this paper because a separate series of papers has been devoted to the coverage of geotechnical ‘actions’.

Table 1 provides a comparison of the coverage of ‘loads’ in BS EN 1991 and BS EN 1998 with BS 5400-2 and HA DMRB BD 37.

Permanent loads (actions) due to self-weight

In BS 5400-2 and HA DMRB BD 37, the design values for the self-weight of materials that are structural elements of a bridge (dead loads), or that impose load on a bridge and are not structural elements (superimposed dead loads such as road surfacing or ballast for example), are based on nominal values for the density of materials given in BS648[20]. Partial factors (γ_{fL}) are applied to the nominal loads, the value of which is dependent upon the accuracy of the load assessment. A reduced partial factor ($\gamma_{fL} = 1.0$) is applied at the Ultimate limit State where the dead load has a relieving effect.

In BS EN 1991-1-1, the ‘self-weight of construction works’ is classified as a ‘permanent action’. For the calculation of self-weight, characteristic values are typically based on the mean material density. The values for typical construction materials are included within Annex A of BS EN 1991-1-1. The self-weight of construction works includes the structural and non-structural elements, fixed services and, the weight of fill, road surfacing and ballast. The total self-weight of structural and non-structural members should be taken into account in combinations of actions as a single action in accordance with BS EN 1991-1-1, clause **3.2(1)**.

In accordance with BS EN 1990 (clause **4.1.2(2)P**), where materials are used with a significant variation of density (due to its source for example) an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$) characteristic value are used to assess the effects of actions. However, where the variability of the permanent action (G_k) can be considered to be insignificant, a single value may be used. This is referred to as the single source principle, which is discussed further in paper 1-2/1.

Load/Classification	BS 5400-2 and HA DMRB BD 37 Section/Clause No.	Action/Classification	Eurocode Section/Clause No.	Eurocode
Dead load – Permanent load	5.1	Self weight - Permanent action	5.2.1	1991-1-1
Superimposed dead load – Permanent load	5.2	Self weight -Permanent action	5.2.3	1991-1-1
Wind – Variable load	5.3	Wind actions – Variable action	3.3	1991-1-4
Temperature – Variable load	5.4	Thermal action – Variable action	2.1	1991-1-5
Effects of shrinkage and creep, residual stresses etc – Permanent load	5.5	Time dependent effects of concrete - Permanent action	Not covered specifically in actions parts	1992-2
Differential settlement – Permanent load	5.6	Geotechnical action	Not covered specifically in actions parts	1997-1
Exceptional loads:	5.7			
Earthquake – Variable load	5.7	Seismic actions	Not covered specifically in actions parts	1998-2
Snow load – Variable load	5.7.1	Snow loads – Variable action	2(1)	1991-1-3
Earth pressure on retaining structures – Permanent load	5.8	Geotechnical action		1997-1
Erection loads – Permanent and variable loads	5.9	Execution actions – Permanent and variable actions	2.1(1)	1991-1-6
Highway bridge live loads – Variable loads	6	Road traffic actions and other actions specifically for road bridges – Variable actions	2.2(1)	1991-2
Foot/cycle track bridge live loads – Variable loads	7	Actions on footways, cycle tracks and footbridges – Variable actions	2.2(1)	1991-2
Railway bridge live loads – Variable loads	8	Rail traffic actions and other actions specifically for railway bridges – Variable actions	2.2(1)	1991-2
Derailment loads– Variable loads	8.5	Derailment actions from rail traffic on a railway bridge – Accidental actions	6.7.1	1991-2
Collision load on supports and superstructures of bridges over railways – Variable load	8.6	Derailment under or adjacent to a structure and other actions for Accidental Design Situations	6.7.2	1991-2 (1991-1-7)
Fatigue – Variable load	Covered in BS 5400-10	Fatigue load models – Variable action Traffic loads for fatigue – Variable action	4.6 6.9	1991-2
Deformation requirements – Permanent/Variable loads	8.8	Serviceability and other Specific Limit States	A2.4.2[(1)] A2.4.3[(1)] A2.4.4[(1)]	1990, Annex A2
Footway and cycle track loading on railway bridges – Variable load	8.9	Actions for non-public footpaths	6.3.7(2)[(2)]	1991-2

Notes

1. *Includes deformation and vibration criteria for road bridges, verification of footbridge vibration and verification of deformations and vibration for rail bridges.*
2. *Includes walkways for maintenance personnel, pedestrian and cycle loads.*

Table 1. Comparison of the coverage of 'actions' and 'loads' in BS 5400-2 and HA DMRB BD 37 with the Eurocodes

BS EN 1991-1-1 includes additional provisions for upper and lower values of self-weight of non-structural bridge elements in section **5.2.3**. These provisions are nationally determined parameters (NDPs) and the values in the UK National Annex to BS EN 1991-1-1[12] are:

i. The upper and lower densities for railway ballast are 21 kN/m^3 and 17 kN/m^3 respectively.
ii. The nominal depth of railway ballast should vary by ± 30% over the top 300 mm depth.
iii. The self-weight of waterproofing, surfacing and coatings for other bridges (e.g. road and footbridges) a deviation from the nominal thickness of ± 40% should be specified where an allowance for resurfacing is included in the nominal thickness and + 55% and – 40% where it is not included.
iv. A deviation of ± 20% should be used for the self-weight of cables, pipes and ducts.
v. The self-weight of handrails, safety barriers, kerbs etc should be taken as the nominal value.

In BS 5400-2 and HA DMRB BD 37, variability in density or thickness of structural and non-structural materials is taken into account by varying the value of the partial load factor (see clauses **5.1.2.1**, **5.1.2.2**, **5.2.2.1** and **5.2.2.2**).

Permanent and variable loads (actions) due to erection

Section 5.9 of BS 5400-2 and HA DMRB BD 37 provides information concerning the loads to be considered during erection. Although it is not explicitly stated, the scope has been taken in practice to mean permanent and variable loads (actions) during construction (and alteration) of bridges, including where relevant, the effects of traffic.

The loads covered in BS 5400-2 and HA DMRB BD 37 are:

i. Temporary loads
ii. Permanent loads
iii. Disposition of permanent and temporary loads
iv. Wind and temperature effects
v. Snow and ice loads

Table 2.1 of BS EN 1991-1-6, summarises the actions and their classification, to be considered during 'execution'. BS EN 1990 defines 'execution' as:

'all activities carried out for the physical completion of the work including procurement, the inspection and documentation thereof

NOTE The term covers work on site; it may also signify the fabrication of components offsite and their subsequent erection on site.'

The actions covered in BS EN 1991-1-6 **Table 2.1** are:

i. Self-weight
ii. Soil movement
iii. Earth pressure

iv. Prestressing
v. Pre-deformations
vi. Temperature
vii. Shrinkage/hydration effects
viii. Wind actions
ix. Snow loads
x. Actions due to water
xi. Atmospheric ice loads
xii. Accidental
xiii. Seismic

BS 5400-2 and HA DMRB BD 37 does not preclude consideration of the actions not mentioned, but no information is provided other than for the loads listed above. BS EN 1991-1-6 is more specific about coverage of actions and is more detailed where coverage is provided.

In BS 5400-2 and HA DMRB BD 37 , traffic loads during construction are not mentioned specifically but that does not mean that they should not be considered and in practice they have. For design using the Eurocodes, there is provision for adjustment of Load Model 1 (LM1) values for transient design situations (i.e. during construction).

Further, actions due to water and atmospheric icing are not currently covered by the Eurocodes. The guidance provided in BS EN 1991-1-6 describes a method for determination of water pressure and, for atmospheric icing, guidance is provided in the UK National Annex to BS EN 1991-1-6[14], for which reference is made to ISO 12494[21].

Permanent loads (actions) due to shrinkage and creep

Section 5.5 of BS 5400-2 and HA DMRB BD 37, covers the sources of strain arising from the nature of construction materials, the conditions of manufacture and the circumstances of fabrication and erection. The requirements for design are contained within the relevant material code parts of BS 5400.

Similarly for the Eurocodes, no specific provisions have been made for shrinkage and creep effects of structural building materials in BS EN 1991. The intention is that they should be determined according to the relevant material Eurocode parts (BS EN1992 for example).

Permanent loads (actions) due to snow

In clause **5.7.1** of BS 5400-2 and HA DMRB BD 37, snow loading should be considered in accordance with local conditions relevant to Great Britain. It is generally ignored for load combinations 1 to 4 i.e. not considered in combinations involving live load, wind or temperature. However, in circumstances where dead load stability is critical (opening and covered bridges for example), consideration of snow loads is necessary.

Clause **1.1(8)** of BS EN 1991-1-3, primarily provides design requirements for building roofs although the scope states that civil engineering works are within scope. However, Clauses **NA.4.1.1** and **NA.4.1.2** of the UK National Annex to BS EN 1991-1-3[13], provide guidance on the application of snow loads to bridges. Snow loads may generally be ignored except where local conditions dictate the need for their consideration, such as for opening and covered bridges and situations where stability due to permanent loads is critical for example.

Actions in BS EN1998

Seismic design of UK bridges - introduction

Before 2009, seismic actions were not generally considered for UK road and rail bridges, although the design of some major estuarial crossings, such as the Kessock Bridge (Cullen Wallace and Nissen 1984[24]) and the Second Severn Crossing (Mizon and Kitchener, 1997[26]) did so to some extent. The situation has changed with the publication in 2009 of the UK National Annexes to BS EN 1998-1[15] and BS EN 1998-2[16] and the accompanying BSI Published Document PD6698. These documents give extensive guidance which has been widely reviewed and discussed by the bridge design and wider structural engineering communities. For the first time, specific guidance is provided on those situations (still expected to be exceptional) where seismic design should be considered and recommendations are given on design seismic ground motions to be used in those situations – effectively the seismic loading requirements. For most bridges, an explicit consideration of seismic actions is not needed and the robustness and lateral strength provided by design for non-seismic actions provides adequate resistance for any low level shaking that may occur. Highways Agency intends to publish in due course an Interim Advice Note (IAN 124[25]) which will adopt the guidance in PD6698 in determining the need for seismic design in highway bridges.

It should be noted that suspension bridges, timber and masonry bridges, movable bridges and floating bridges are beyond the scope of BS EN 1998-2, although a number of aspects of the code will still be applicable, including the derivation of seismic ground motions.

Seismic analysis

The complexity of the analysis model required for seismic actions will depend on the importance of the bridge (based on its Importance Class, I) and the complexity of the structure. Response spectrum analysis is the default method, while ‘fundamental mode’ (equivalent static) analysis may be used for simple cases, and time history or non-linear static (pushover) analysis may be appropriate for more complex cases. For design using the response spectrum method, the bridge may be analysed separately for each translational component (longitudinal, transverse and, where required, vertical) and the resulting action effects combined by a method such as the SRSS (square root sum of squares) of the three orthogonal components. Where a non-linear time-history analysis is undertaken, the action effects from the three components should be considered to act simultaneously.

Seismic ground motions

The universally acknowledged low level of seismic hazard in the UK is recognised by the UK National Forewords to all parts of BS EN 1998, which state:

‘There are generally no requirements in the UK to consider seismic loading, and the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998 need not apply. However, certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions. It is the intention in due course to publish separately background information on the circumstances in which this might apply in the UK.’

The 'background information' promised in the last sentence of the Forewords was subsequently published as PD 6698. Advice on the circumstances in which seismic design should be considered are discussed in a companion paper (Booth et al 2010, paper 1-3/6); in brief, it is not recommended for the majority of bridges, but may be warranted for some (but not all) critical bridges of Importance Class III, for example where loss of functionality would have a major regional or national economic impact. Seismic design is not considered necessary for UK bridges of Importance Class I or II.

PD 6698 also provides advice on design ground motions. To place the recommendations in context, BS EN 1998-1 recommends that structures of 'normal importance' (Importance Class II) should be designed for ground shaking with a reference return period of 475 years for the no-collapse requirement (broadly equivalent to the ultimate limit state). Structures with higher consequences of failure are then designed for longer return periods by applying an Importance Factor I to the ground motions; the recommended value of I for Importance Class III bridges is 1.35. However, both the reference return period and the values of Importance Factor are subject to National Choice. For reasons discussed by Booth and Skipp (2008)[23], a much longer return period than 475 years is required in areas of low seismicity than for areas of moderate to high seismicity. The longer return period is needed to provide levels of structural reliability under seismic actions equivalent to those in high seismicity areas. The UK National Annex to BS EN 1998-2 provides that, in the absence of a project specific requirement, a reference return period of 2,500 years should be adopted for design seismic actions on UK bridges, with the Importance Factor I taken as 1.0 for Importance Class III bridges. These actions relate to the ultimate limit state. Consideration of ground motions with a much shorter return period for a serviceability limit state is also required in areas of moderate and high seismicity; however, it is not considered necessary in the UK, because this limit state is very unlikely to govern design.

PD 6698 provides a map of peak ground accelerations (PGAs) on rock in the UK for a 2,500 year return period, which is reproduced here as Figure 1. The figure is based on work carried out over many years at the British Geological Survey, Edinburgh, and its basis is reported by Musson and Sargeant (2008) [27]. It can be seen that most of the UK, and all of Northern Ireland, is assigned as having extremely low seismicity (less than 6%g PGA for 2,500 year return period), but parts of Central and Eastern Scotland, Central England, South Wales and North West Wales are relatively more seismic, with a maximum PGA of 18%g for a 2,500 year return period assessed for the region around Bangor.

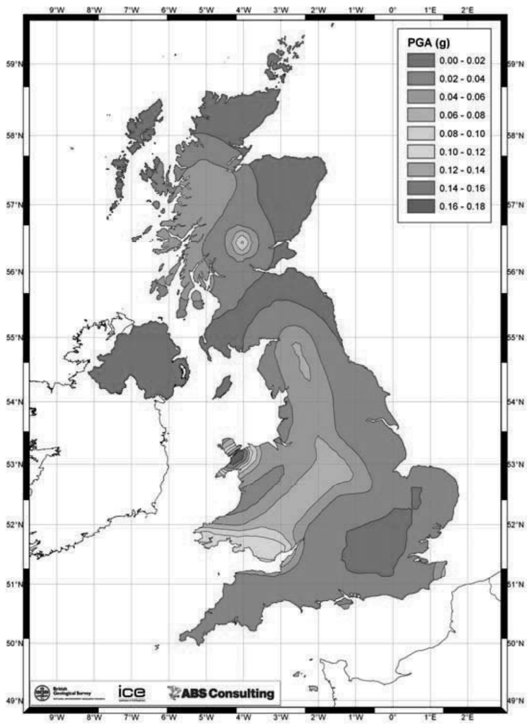

Figure 1. Seismic hazard map of 2 500 year return period Peak Ground Acceleration (PGA) on rock, from Musson and Sargeant (2008)

A rigid body on rock would experience a peak lateral force equal to the PGA times its mass. For example a PGA of 18%g would give rise to a peak lateral force of 18% of the rigid body's weight. However, a number of adjustments are required to arrive at realistic lateral design forces. Firstly, flexible structures respond dynamically to earthquake motions and may amplify or attenuate the peak ground motions, depending on how closely the structure's natural periods of vibration match the predominant periods of vibration of the ground motion. This is allowed for in the design response spectrum provided in most seismic codes of practice, which provide a plot of response against the period of vibration of the structure (the 'structural period' in figure 2). The UK National Annex to BS EN 1998 specifies that the spectra recommended by BS EN 1998-1 for low seismicity areas (Type 2 spectra) should be used in the UK, with the exception (shown in Figure 2) that the transition from the 'constant velocity' to 'constant displacement' sections of the spectra is omitted (Booth and Skipp, 2008).

A second adjustment is needed to allow for the modification, by the superficial soils at a site, to the seismic waves arriving from the underlying bedrock. This modification can be highly significant; softer soils tend to increase both the amplitude and long period of vibration content of the motions. BS EN 1998 gives recommended adjustments to the rock spectra for four standard ground types, ranging from stiff or very dense, to loose or soft soils and the UK National Annex to BS EN 1998 adopts the recommended values.

A third adjustment may be made to allow for the ability of ductile structures to dissipate some of the seismic energy imparted to it by means of plastic yielding. BS EN 1998 recognises this effect by means of the behaviour factor, q, which effectively acts as a divisor of the ground motion response spectra. Ductile structures designed for high q factors need reduced lateral strength, compared with less ductile ones, but with the penalty of more stringent seismic detailing requirements. In areas of low or very low seismicity, such as the UK, it is recommended that seismic detailing measures may be minimal, as discussed in the companion paper (Booth et al, 2010, paper 1-3/6), but that q factors for bridge superstructures should be correspondingly low (i.e. conservative), typically in the range 1.2 to 1.5.

Figure 2 shows the resulting design response spectra for an area with a 2,500 year PGA on rock of 10%g; Figure 1 shows that most of the UK has much lower levels of seismic hazard. A q factor of 1.5 has been assumed, and the spectra shown represent a rock site (BS EN 1998 soil type A) and a site with loose to medium cohesionless soil or soft to firm cohesive soil (BS EN 1998 soil type D). Stiff bridges of spans up to 20m may have periods close to the peak of the spectra. Periods of the fundamental modes of vibration of flexible longer span bridges will be well beyond the peak, although higher modes may be closer to resonance with the ground motion periods of vibration.

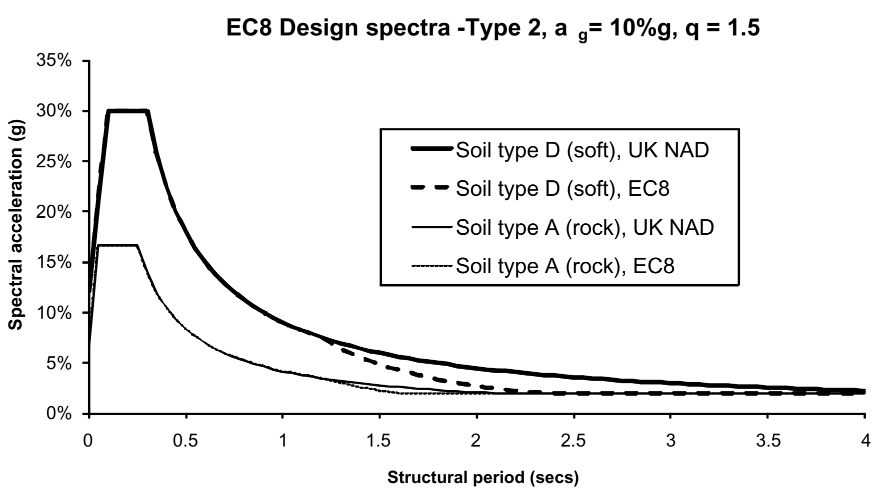

Figure 2. Typical EC8 bridge design spectra for a high consequence of failure bridge in a UK area of above average seismicity

Instead of the use of the PGAs of Figure 1 and the BS EN 1998-1 standard spectral shapes, the UK National Annex to BS EN 1998-1 and PD 6698, permit a site specific determination of the response spectrum at the bridge site, using a return period appropriate to the consequences of failure of the bridge. Because of the limitations, discussed by Musson (2010) [28], of a general seismic zoning map such as Figure 1, particularly at long return periods, site specific determination may be appropriate for very important bridges. It may also be appropriate where the consequences of using Figure 1 result in significantly increased construction costs; this would enable a better appreciation of whether the additional construction costs were justified by the reduction in seismic risk.

Vertical ground motions

The discussion so far has applied only to horizontal ground motions. In areas of moderate to high seismicity, BS EN 1998-2 requires that vertical motions should be considered in certain circumstances. However, vertical motions never need to be considered in areas of low seismicity and so are not an issue for the UK.

Conclusions

The use of the terms ‘actions’ and ‘loads’ has paper has been examined and the rationale behind the adoption of the generic term ‘actions’ in the Eurocodes, has been explained.

The coverage of ‘actions’ (‘loads’) in the Eurocodes and the former British Standards (predominantly BS 5400-2 and HA DMRB BD 37) has been compared. The differences in design practice have been described for BS EN 1991-1-1, BS EN 1991-1-3 and BS EN 1991-1-6. The other action parts are covered in separate papers.

The seismic design requirements do not have a precedent in the UK and the provisions for derivation of ‘actions’ (‘loads’) in the UK National Annex to BS EN 1998 and the supporting guidance in PD 6698, have been described. There are, however, particular issues which will need ongoing review. The suitability and impact on design of using the zoning map (Figure 1) will need to be established, and the design and detailing recommendations will need to be monitored and if necessary modified after a suitable period of use. Experience of use is likely to grow slowly as only large or important bridges are likely to be designed to resist seismic effects.

Despite best endeavours, it is highly likely that inconsistencies and lack of clarity remain in the Eurocodes. Although some of these have been identified through the development phases for the Eurocodes and their National Annexes, the most effective source for identification of issues with the Eurocodes, will be the experience of those tasked with implementation of the Eurocodes for the design of bridges. The BSI committees responsible for Eurocode and National Annex development, welcome feedback on problems encountered. In some cases, the issues may already be known and advice be possible but in other cases their may be a need for further work to improve the Eurocodes. Feedback through the appropriate forums for dealing with issues arising from the implementation of the Eurocodes is encouraged.

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THE UK NATIONAL ANNEX TO BS EN 1991-1-4, BS EN 1991-1-5, AND PD 6688-1-4

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Abstract

The objective of this paper is to give the background to the development of the National Annexes to BS EN 1991-1-4: 2005[19], BS EN 1991-1-5: 2003[20] and Published Document PD 6688-1-4: 2009[21], focussing particularly on those aspects which relate to the design of bridges. The relevant clauses in the Eurocodes have been reviewed, comparing them against clauses in the UK bridges standard BS 5400[22] and any other relevant British Standards.

Introduction

The National Annex (NA) to a Eurocode may give information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters (NDPs). It may also contain decisions on the application of informative annexes and references to non-contradictory complementary information.

The paper gives reasons for the choice of the NDPs contained in the UK NAs to BS EN 1991-1-4: 2005[23] and BS EN 1991-1-5: 2003[24]. It also makes comparisons with the equivalent clauses in BS 5400[22] and other relevant Standards.

BSI has published a series of Published Documents (PDs) giving non-contradictory and complementary information for use in the UK with a Eurocode and its NA. PD 6688-1-4: 2009[21] was published to supplement BS EN 1991-1-4: 2005[19] and its NA[23].

For convenience the format of the relevant National Annex / Published Document has been adopted in respect of the heading for the clauses.

UK National Annex to Eurocode 1 – Actions on Structures Part 1-4: General Actions – Wind Actions

Background

The process of drafting BS EN 1991-1-4: 2005[19] was extremely difficult; there was much opposition and criticism of the draft wind Eurocode (ENV1991-2-4)[25] and the project team converting that draft to the pre-standard and standard were faced with conflicting opinions and recommendations from Member States' representatives. As a result there are many clauses in BS EN 1991-1-4: 2005[19] that allow National Choice, through Member States' NAs. In fact there are 50 clauses where Nationally Determined Parameters or alternative procedures are allowed, thereby resulting in an exceptionally long and comprehensive NA. Decisions also needed to be made as to whether to adopt or replace six informative annexes, and how to introduce non-contradictory complementary information (NCCI).

General

The major aspects of the NA[23] relate to the wind structure, as defined in section 4 of BS EN 1991-1-4: 2005[19] and in which twelve clauses require National Determined Parameters or alternative procedures.

The Eurocode uses the peak factor model, common with most wind loading Codes. However the Eurocode linearises the basic equation relating pressure to velocity whereas the UK NA to BS EN 1991-1-4[23] uses the full relationship for peak loads to maintain the required factor of safety for low-rise buildings in urban areas.

The basic wind speed in BS EN 1991-1-4: 2005[19] is the 10 minute mean wind velocity in 'open country' (as opposed to the hourly mean wind velocity used in the UK Code BS6399 Part 2[26]). The velocities obtained from the resulting wind map are thus higher, by a factor of 1.06, compared with the BS6399 Part 2[26] map to account for this difference in averaging times. In addition advantage has been taken in the UK NA to EN1991-1-4[23] of increased source data thereby modifying the isopleths in the map for wind speeds in the UK.

The basic wind speed is adjusted for altitude and topography (termed orography in the Eurocode), in the UK NA to EN1991-1-4[23] the former being a more logical formulation than in the UK code and the latter being identical to that in BS6399 Part 2[26]. The UK NA to BS EN 1991-1-4[23] also provides directional factors, seasonal factors and probability factors – all compatible with the equivalent factors in the UK Code[26].

The UK NA to BS EN 1991-1-4[23] treats terrain roughness differently to BS EN 1991-1-4: 2005[19] in that the Eurocode specifies 5 specific roughness categories (ranging from 0 for sea or coastal areas to IV for urban areas) whereas the UK NA to EN 1991-1-4 adopts the UK procedure of defining just 3 categories (sea, country and town) but with due allowance for the development of the appropriate wind structure dependent on the distance downstream of the site from the sea, or for town terrain the distance of the site into the town.

Graphical presentation of the exposure factor ($c_e(z)$) is then provided in the UK NA to EN 1991-1-4[23].

The UK NA to BS EN 1991-1-4[23] has separated the size factor from the dynamic factor, as allowed in BS EN 1991-1-4: 2005: **Clause 6.1 (1)**. This provides a better means of treating individual components in a structure. In using the detailed method provided in BS EN 1991-1-4: 2005 [19] the UK NA to BS EN 1991-1-4[23] adopts Annex B as the approach to use to determine dynamic response. It should be noted however that the given procedure only applies to the along wind response of a structure in the fundamental mode, the mode shape having a constant sign. Thus this is of limited use for bridges, other than for towers, piers (during construction, prior to the deck being attached) and cantilevers.

BS EN 1991-1-4: 2005: **Section 7** deals with pressure and force coefficients and generally relates to building type configurations; much of the information contained in this section was based on the work undertaken at the Building Research Establishment and used in the United Kingdom code, BS6399 Part 2[26].

BS EN 1991-1-4: 2005: **Section 8** deals specifically with wind actions on bridges and NDPs can be given in 11 clauses. These are discussed in turn below.

NA.2.42 – Wind Action of other types of bridges [8.1(1) Note 1]

The scope of BS EN 1991-1-4: 2005[19], in dealing with wind actions on bridges, is limited to self supporting bridges of constant depth and with cross-sections of common forms as in BS EN 1991-1-4: 2005 **Figure 8.1**. Cable supported bridges, arch bridges, roofed bridges are excluded. Also excluded are bridges with multiple or significantly curved decks.

The BS EN 1991-1-4: 2005[19] also gives no guidance on deck vibrations from transverse wind turbulence or vibrations where more than the fundamental mode needs to be considered but does allow, however, for additional guidance on all these matters to be given in the UK NA to BS EN 1991-1-4[23].

It has not been deemed possible to codify all these aspects in the UK NA to BS EN 1991-1-4[23] and guidance has been limited to recommending the use of the BS EN 1991-1-4: 2005[19], in conjunction with the NA[23], for wind actions on elements of those bridges outside the scope of the Eurocode, but to seek specialist advice in deriving their overall response to wind actions.

NA.2.43 – Angle of the wind direction relative to the deck axis [8.1(1) Note 2]

Wind actions on bridges produce forces in the *x*, *y* and *z* directions as shown in BS EN 1991-1-4: 2005: **Figure 8.2**, reproduced here as Figure 1.
where:
x-direction is the horizontal direction, perpendicular to the span
y-direction is the horizontal direction along the span
z-direction is the vertical direction perpendicular to the plane of the deck

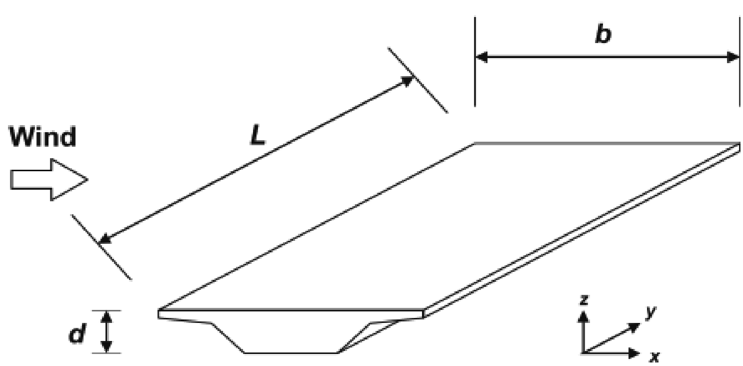

Figure 1. Directions of wind actions on bridges

It should be noted that the notation used for bridges differs from that defined for building structures in BS EN 1991-1-4[19]. This is to ensure that conventional notation in the bridge field is maintained (i.e. the depth of the bridge deck is defined as 'd') Thus the following notations are used for bridges:
L length in *y*-direction
b width in *x*-direction
d depth in *z*-direction

The values to be given to *L*, *b* and *d* in various cases are, where relevant, more precisely defined in further clauses. *It is important to recognise that when Sections 5 to 7 are referred to, the notations for b and d need to be readjusted.* The UK NA to BS EN 1991-1-4[23] refers to BS EN 1991-1-4: 2005: **Figures 8.2 and 8.6** to clarify how the axes are defined.

NA.2.44 – Fundamental value of the basic velocity to be used when considering road traffic simultaneously with the wind [8.1(4)]

Limiting wind speeds with both road and rail traffic have been implied in the UK NA to BS EN 1991-1-4[23], expressed as an upper limit to the value of the peak velocity pressure $q_p(z)$, at the level of the deck.

For road traffic the limit has been taken as 35m/sec, compatible with current UK practice, leading to a limit to $q_p(z)$ of: ½ x 1.226 x 35^2 = 750 Pa as stated in the UK NA to BS EN 1991-1-4[23].

NA.2.45 – Fundamental value of the basic velocity to be used when considering railway traffic simultaneously with the wind [8.1(5)]

For rail traffic the limiting wind speed has been taken as 40m/sec based on the criteria for prevention of damage to overhead line electrification equipment, and the risk of trains overturning. This leads to a limit to $q_p(z)$ of: ½ x 1.226 x 40^2 = 980 Pa as stated in the UK NA to BS EN 1991-1-4[23].

NA.2.46 – Choice of the response calculation procedure [8.2(1) Note 1]

BS EN 1991-1-4: 2005: **Clause 8.2** allows the NA[23] to specify when a dynamic response procedure is needed for bridges. A note (Note 3) suggests that road and railway bridges of less than 40m span do not normally require a dynamic response procedure.

The UK NA to BS EN 1991-1-4: 2005: **NA.2.46.1**, extends this relaxation further and stipulates that highway and railway bridges up to 200m span do not normally require explicit allowance for dynamic response in the wind direction and, in clause **NA 2.46.2**, states that such effects can be ignored in the vertical direction as well, provided the fundamental frequencies in bending and torsion are greater than 1 Hz or a dynamic magnification parameter is less than unity.

This parameter is identical, other than in presentation, to the corresponding parameter in BD 49/01[27] apart from the difference in the basic mean wind speed (see below).

Thus in BD 49/01[27] the aerodynamic magnification parameter, P_T, is given by:

$$P_T = \left(\frac{\rho b^2}{m}\right)\left(\frac{V_s}{f_B b}\right)^2 \frac{\sigma_{flm} b}{\sigma_c}$$

The equivalent parameter in the NA[23] (equation NA.5) is:

$$P(z)\frac{\sigma_{fm}.b}{\sigma_c}$$

where $P(z) = \left(\frac{v_m(z)}{n_b.b}\right)^2\left(\frac{\rho b^2}{m}\right)$

The suggested values of σ_{flm} (σ_{fm}) and σ_c are the same in each document.

Thus the two parameters are identical apart from the use of V_s in BD 49/01[27] and $v_m(z)$ in the NA[19]. In the former case the wind is an hourly mean wind speed and in the latter it is a ten minute average. Thus for the UK NA to BS EN 1991-1-4[23], the magnification factor will be 12% higher for a given site than in BD 49/01[27]. As the criteria is for the parameter to be less than unity the UK NA to BS EN 1991-1-4[23] will be 6% more conservative, bridges that may be just acceptable to the BD may require a dynamic response analysis when using the NA[23].

Both the simplified procedure for single span bridges (see NA.2.49) and a procedure for continuous bridges (see NA.2.53) are given in the UK NA to BS EN 1991-1-4[23].

The UK NA to BS EN 1991-1-4: 2005: **NA.2.46.3** then proceeds to provide the criteria for when aerodynamic stability effects, such as vortex excitation, galloping and flutter need to be considered. An aerodynamic susceptibility parameter is derived which is the same as that provided in BD 49/01[27], duly adjusted for the differing wind structure adopted in the Eurocode. If the parameter is low (< 0.04) then aerodynamic effects may be considered insignificant; if the parameter is high (> 1.0) then wind tunnel tests are required; between these extremes the provisions and criteria set out in PD 6688-1-4: 2009[21] should be satisfied.

It has been suggested that for clarity the words *"and whether dynamic response procedures are needed"* in the first paragraph of UK NA to BS EN 1991-1-4: 2005: **NA.2.46.3** be deleted as they could be confused with in-line turbulence response which is not the case.

NA.2.47 – Force coefficients for parapets and gantries on bridges [8.3(1)]

The NA[23] recommends the use of the force coefficients provided in BS EN 1991-1-4: 2005: **Clauses 7.4, 7.6, 7.7, 7.9 and 7.11** to determine the appropriate coefficients for parapets and gantries.

NA.2.48 – Reduction in drag co-efficient for F_w [8.3.1(2)]

Whilst the reduction factor for inclined webs of box girder bridges is allowed within the general method of BS EN 1991-1-4: 2005: **Clause 8.3.1** and NA[23] this cannot be extended to the simplified procedure in BS EN 1991-1-4: 2005: **Clause 8.3.2.**

NA.2.49 – Values of the wind load factor C [8.3.2(1)]

The simplest procedure for determining wind actions on bridges in BS EN 1991-1-4: 2005[19] is to use clause 8.3.2, provided it can be shown that a dynamic response procedure is not necessary (see UK NA to BS EN 1991-1-4: 2005: **Clause NA.2.46.2**). Assuming that a simple static analysis under lateral loads is all that is required then BS EN 1991-1-4: 2005: **Equation 8.2** can be used. Equation 8.2 is:

$$F_W = 0{,}5 \cdot \rho \cdot v_b^2 \cdot C \cdot A_{\text{ref,x}}$$

C is a wind load factor taken as the product of the force (drag) coefficient c_{fx} and the exposure factor $c_e(z)$

It is not clear however in BS EN 1991-1-4: 2005[19], and an amendment is probably desirable to amend this, that, provided a dynamic response procedure for the assessment of vertical wind response is not needed (see UK NA to BS EN 1991-1-4: 2005: **NA.2.46.2** for the criteria) then the procedure of BS EN 1991-1-4: 2005: **8.3.2** should be followed and in the general case c_e should be calculated according to UK NA to BS EN 1991-1-4: 2005: **NA.2.17** (over-ruling 4.5 of the Eurocode) and $c_{f,x}$ should be calculated according to BS EN 1991-1-4: 2005: **8.1**. The use of the simplification C in BS EN 1991-1-4: 2005: **8.3.2** is generally an upper bound solution to the product of c_e and $c_{f,x}$ and this should be made clear.

Values of *C* are given in the UK NA to BS EN 1991-1-4: 2005: **Table NA.7** and were derived as upper bound values using BS EN 1991-1-4: 2005: **Figure 8.3** for the drag coefficient from the Eurocode and Figure NA.7 for $c_e(z)$ from the NA. In this simple approach no reduction is considered for loaded lengths where lack of correlation of the wind gusts will reduce the quasi-static loads. For continuous bridges and bridges of longer spans this is conservative and reductions are allowed as shown in UK NA to BS EN 1991-1-4: 2005: **NA.2.53**.

In the UK NA to BS EN 1991-1-4: 2005[23], $c_e(z)$ varies with distance of the site from the sea and whether the site is in 'town' terrain. Assuming, conservatively in this case, that the site is in country terrain values of *C* have been derived for sites:

a) 0,1 km from the sea
b) 10km from the sea, and
c) 100km from the sea

These values are shown in Table 1 below:

b/d_{tot}	$z_e \leq 20$m			$z_e = 50$m		
	a	b	c	a	b	c
$\leq 0{,}5$	7,7	7,1	6,7	8,9	8,7	8,2
>4,0	4,2	3,9	3,6	4,8	4,7	4,4

Table 1. Derivation of wind load factor *C* in the NA

It can be seen from Table 2 that the values adopted for the UK NA to BS EN 1991-1-4[23] are reasonable upper bounds to the most severe of the derived figures.

In order to compare these figures with the equivalent values in BD37/01[28] it is necessary to correct the BD values as these are based on an hourly mean wind speed whereas BS EN 1991-1-4: 2005[19] is based on a ten minute averaging time. Thus the BD values need to be factored by $(1/1{,}06)^2$. i.e. if the 10 minute wind speed map is used for comparison in both Codes then the map wind speed for the EN is 1.06 x the map wind speed for the BD.

As can be seen from Figure 2 in which the curves from the Codes for the force (drag) coefficient are shown for solid faced bridges, there are some differences.

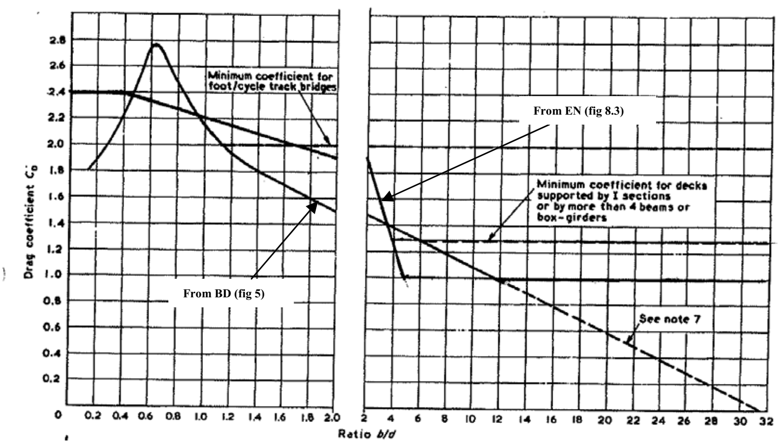

Figure 2. Comparison of drag coefficients for superstructures with solid elevation

Associated with BD37/01: **Figure 5** and BS EN 1991-1-4: 2005: **Figure 8.3** are notes qualifying the scope and application of these figures. When these are compared it can be seen that generally the two documents provide similar guidance.

Table 2 shows the equivalent maximum values of *C* derived from BD37 and the NA respectively:

b/d_{tot}	$z_e \leq 20$m		$z_e = 50$m	
	EN-NA	BD37	EN-NA	BD37
≤ 0,5*	7,4	8.0	9,1	9.4
>4,0	4,0	4.1	4,9	4.8

** Figure 5 in the BD gives a peak value of C_D when b/d_{tot} is 0.65. This peak figure has been used in this table*

Table 2. Comparison between the NA and BD for equivalent values of the factor *C*

From this comparison it can be seen that the simple method in the EN compares closely with the equivalent derived values from the BD.

However BD37/01[28] contains an upper bound value of pressure to be applied to the vertical projected area of the bridge of 6kN/m^2. Clearly this applies to all sites in the UK, being independent of the wind speed. Application of the NA[23] to the seven sites chosen for the calibration study for DCLG for buildings shows that this value is a very conservative figure as may be seen from Table 3.

site	Location	altitude (m)	map wind speed to NA (m/sec)	Altitude factor			Wind speed v_b (m/sec)	Wind load to Eq 8.2 of BS EN (kN/m^2)			
height above ground z (m)				10	20	50	10	20		50	
C factor								C = 7.4	C = 4.0	C = 9.1	C = 4.9
A	London	10	21.5	1.01	1.01	1.01	21.72	2.14	1.16	2.63	1.42
B	Birmingham	124	21.8	1.124	1.11	1.09	24.50	2.72	1.47	3.35	1.80
C	Glasgow	30	25.5	1.03	1.03	1.02	26.27	3.13	1.69	3.85	2.07
D	Scarborough	60	22.8	1.06	1.05	1.04	24.17	2.65	1.43	3.26	1.75
E	Brighton	50	21.7	1.05	1.04	1.04	22.79	2.35	1.27	2.90	1.56
F	Haverford West	50	24.2	1.05	1.04	1.04	25.41	2.93	1.58	3.60	1.94
G	Sheffield	292	22.2	1.292	1.25	1.21	28.68	3.73	2.02	4.59	2.47

Table 3. Static load for short span bridges obtained from NA for seven sites

Clearly the 6kN/m^2 has to apply to all sites in the UK and so for a site in the Western Isles of Scotland where the basic wind speed is, say, 30m/sec the highest value of C would result in a pressure of about 5kN/m^2. The upper bound value of 6kN/m^2 has not been adopted in the National Annex.

It can also be seen that the use of the altitude factor in the UK NA to BS EN 1991-1-4: 2005[23] (that reduces with height above the ground) will provide lower values to the pressure than those used previously.

For truss girder bridges the BS EN 1991-1-4: 2005[19] only provides sparse information. Force (drag) coefficients for single trusses are given in BS EN 1991-1-4: 2005[19] and these compare closely, at least for trusses formed of flat-sided members, with the values given in BD37 **Table 6**.

NA.2.50 – Value of the force coefficient, $c_{f,z}$ [8.3.3(1) Note 1]

The force coefficients $c_{f,z}$ set out in BS EN 1991-1-4: 2005[19] are identical to those adopted in BD37, based on a limited study of wind tunnel tests in the UK. Accordingly these values have been accepted in the UK NA to BS EN 1991-1-4: 2005[23].

NA.2.51 – Value of the force coefficient, $c_{f,y}$ [8.3.4(1)]

The UK NA to BS EN 1991-1-4: 2005[23] expands the requirements of BS EN 1991-1-4: 2005[19] for longitudinal wind effects on a bridge adopting the procedure of BD37/01[28] in dealing with this aspect.

NA.2.52 – Simplified rules for wind effects on bridge piers [8.4.2(1) Note 1]

Values in BS EN 1991-1-4: 2005[19] for force coefficients on piers can be derived from the data given in Section 7, but are difficult to apply and are not comprehensive. Accordingly the UK NA to BS EN 1991-1-4: 2005[23] contains a table setting out force coefficients for most commonly used cross sections – and including slenderness effects by providing the values over a range of height to breadth ratios. In a subsequent amendment to the UK NA to BS EN 1991-1-4: 2005[23], the Table was reviewed and minor changes made. The amended values are shown in Table 4, where they are compared with the equivalent values from BD 37/01[28]. It can be seen that the values are broadly similar.

In Note 2 to BS EN 1991-1-4: 2005: **8.4.2**, reference is made to torsional effects on piers and in the NA the procedure set out in UK NA to BS EN 1991-1-4: 2005: **NA2.23** is recommended. In this it is intended that this treatment is used to account for torsional effects due to perhaps spreader beams on cantilevered piers or temporary works on the piers. Generally it would be inapplicable where overall loads on the piers are considered.

PLAN SHAPE	H/B		1	2	4	6	10	20	40
	t/b								
	0.25	EN-NA	1.31	1.37	1.43	1.49	1.61	1.76	1.92
		BD37	1.3	1.4	1.5	1.6	1.7	1.9	2.1
	0.333	EN-NA	1.36	1.43	1.49	1.56	1.68	1.84	2.00
		BD37	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	0.50	EN-NA	1.44	1.51	1.58	1.65	1.78	1.95	2.12
		BD37	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	0.667	EN-NA	1.50	1.57	1.65	1.72	1.85	2.03	2.21
		BD37	1.3	1.4	1.5	1.6	1.8	2.0	2.2
rectangular	1	EN-NA	1.35	1.42	1.48	1.54	1.66	1.83	1.99
		BD37	1.2	1.3	1.4	1.5	1.6	1.8	2.0
t	1.5	EN-NA	1.17	1.23	1.28	1.34	1.44	1.58	1.72
		BD37	1.0	1.1	1.2	1.3	1.4	1.5	1.7
b	2	EN-NA	1.04	1.09	1.14	1.19	1.28	1.41	1.53
		BD37	0.8	0.9	1.0	1.1	1.2	1.3	1.4
→	3	EN-NA	0.86	0.90	0.94	0.98	1.06	1.16	1.26
wind		BD37	0.8	0.8	0.8	0.9	0.9	1.0	1.2
	4	EN-NA	0.73	0.77	0.80	0.83	0.90	0.99	1.07
		BD37	0.8	0.8	0.8	0.8	0.8	0.9	1.1
square on diagonal		EN-NA (uses BD37)	1.00	1.10	1.10	1.20	1.20	1.30	1.40
		BD37	1.0	1.1	1.1	1.2	1.2	1.3	1.4
Octagonal	a)	EN-NA	0.69	0.73	0.76	0.79	0.85	0.94	1.02
	b)	EN-NA	0.82	0.86	0.90	0.94	1.01	1.11	1.20
12 sided polygon		EN-NA	0.69	0.73	0.76	0.79	0.85	0.94	1.02
		BD37	0.7	0.8	0.9	0.9	1.0	1.1	1.3
circle with smooth surface $tV>6m^2/s$		EN-NA	0.44	0.46	0.48	0.50	0.54	0.60	0.65
		BD37	0.5	0.5	0.5	0.5	0.5	0.6	0.6
circle with smooth surface $tV\leq 6m^2/s$		EN-NA	0.76	0.79	0.83	0.86	0.93	1.02	1.11
		BD37	0.7	0.7	0.8	0.8	0.9	1.0	1.2

a) smooth surface r/b greater than > 0.075; Re > 7 . 10^5

b) smooth surface r/b greater than ≤ 0.075; Re > 3 . 10^5

Table 4 Comparison of force coefficients for piers between BS EN 1991-1-5 with UK NA and BD 37/01

NA.2.53 – Quasi-static procedure for along wind effects

BS EN 1991-1-4: 2005[19] contains no provisions for dealing with continuous bridges so the methodology adopted in BD37/01[28] has been adopted in the UK NA to BS EN 1991-1-4: 2005[23], allowing for peak wind loads on adverse areas of the bridge and application of the wind actions from the 10 minute mean wind speed where the wind reduces the wind effects.

It should be noted that there continues to be a typographic error in the UK NA to BS EN 1991-1-4: 2005: **NA.2.53 a)** where reference to (2) and (3) should be to b) and c). This will be corrected in a planned amendment to the document.

Annex E – Vortex shedding and aeroelastic instabilities

BS EN 1991-1-4: 2005: **Annex E** covers 'vortex shedding and aeroelastic instability' but its scope is limited to building structures and chimneys. No criteria are provided to deal with the aerodynamic stability of bridges and this was considered to be an unacceptable omission for the UK where BD49/01 contains guidance on such effects. Accordingly it was decided not to adopt BS EN 1991-1-4: 2005: **Annex E** but to replace it with a NCCI document, which would contain (i) all the information in the current Annex E, and (ii) additional information on the aerodynamic response of bridges, as contained in BD49/01[27].

PD6688-1-4: 2009[21] now contains this non-contradictory complementary information. It uses the basic parameters and criteria for vortex shedding as contained in Annex E, but augmented with the appropriate Strouhal numbers for bridge sections.

Cross wind amplitudes for bridges can be calculated using the procedures from BD49/01[27], whereas for other structures the Annex E procedure is retained.

Values of damping, as provided in Annex F, are recommended, in the absence of measured data, and the dynamic sensitivity parameter, K_D, related to the effects of vortex shedding, contained in BD49/01[27] has been introduced into PD6688-1-4: 2009[21].

Galloping of flexible structures is considered in BS EN 1991-1-4: 2005: **Annex E** and these procedures are contained in PD6688-1-4: 2009[21]. In addition the procedures for predicting the onset of galloping and stall flutter and for classical flutter for bridge decks, and the criteria to be satisfied, as contained in BD49/01[27], are included in the document.

PD6688-1-4 – Background Information to the National Annex to BS EN 1991-1-4 and Additional Guidance

General

There are no matters specific to bridges in the main body of PD 6688-1-4: 2009[21] other than background to the basic wind parameters including the basic wind velocity, roughness, altitude, orography and turbulence factors.

Annex A (informative) – Vortex shedding and aeroelastic instabilities

Annex E of the Eurocode deals with vortex shedding and aeroelastic instabilities. The main reason for not permitting it in the UK is that it contains no specific information for such responses for bridges. An alternative version that may be used in the UK is given in PD 6688-1-4: 2009: **Annex A**. This incorporates the provisions of BD49/01[27], duly amended for the notation and different wind structure (10 minute mean wind speeds in the Eurocode rather than hourly wind speeds as used in BD37/01[28] and BD49/01[27]).

UK National Annex to Eurocode 1 – Actions on Structures

Part 1-5: General actions – Thermal actions

Consistent with much of the Eurocode philosophy, the discussion of the representation of thermal actions within section 4 of EN 1991-1-5[20] is based on the most general case of a full 3 dimensional temperature field applied to a prismatic element. Figure 4 in EN 1991-1-5[20] identifies four discrete components:

a) A uniform temperature component, ΔT_u.
b) A linearly varying temperature difference component about the vertical member axis ΔT_{My} (temperature varies with y).
c) A linearly varying temperature difference component about the horizontal member axis ΔT_{Mz} (temperature varies with z).
d) A non linear temperature difference component that results in a system of self equilibrating residual stresses ΔT_E.

In practice reduced sub sets of these components are used during design, depending on the form and function of the structure in question.
For bridges the most important components are:

i. the ΔT_u component, which results in axial expansion/contraction and is a function of changes in shade air temperature. This was termed the effective bridge temperature within BS 5400-2[29]/BD 37/01[28];
ii. the ΔT_{Mz} component, which results in a vertical bending effect about the horizontal member axis and is a function of heat gain/loss from the top or bottom surfaces of the deck (typically incoming solar radiation or re-radiation during the night);
iii. the ΔT_E component, which results in a set of self equilibrating residual stresses. The different material specific parts (EN 1992, EN 1993 etc give advice on how the resulting stresses should be dealt with during design).

The combination of ΔT_{Mz} and ΔT_E was termed the temperature difference within BS 5400-2[29]/BD 37/01[28].

NA.2.2 – Bridge deck types [6.1.1(1)]

The Eurocode limits its scope to three basic types of bridge deck section:

a) Type 1: steel decks	- with steel box girders
	- with steel truss or plate girders
b) Type 2: composite decks	
c) Type 3: concrete decks	- with concrete slabs
	- with concrete beams
	- with concrete box girders.

These essentially cover the four ‘Groups’ identified within BD37/01[28] (the steel groups 1 and 2 have been amalgamated to simplification).

However it was considered in drafting the NA that buried concrete box and portal frame structures (previously covered in BD31/01[30]), and masonry arch bridges with solid spandrels (previously covered in BD91/04[31]) should be included.

Although the thermal characteristics of buried concrete box and portal frame structures, and of masonry arch bridges would be expected to be similar, the treatment of thermal actions in BD31/01[30] and BD91/04[31] was rather different. There appears to be very limited research on temperature in buried structures, and no evidence to justify such a difference of approach. It was therefore decided that the NA should adopt a common approach to the treatment of buried concrete box and portal frame structures and for masonry arch bridges (particularly since the masonry might be concrete).

The approaches in BD 31/01[30] and BD 91/04[31] were reviewed and it was decided to draw more upon the BD 91/04[31] approach in drafting the NA. It had been recognised for some time that there is little physical justification for the approach to treating temperature difference included in BD 31/01[30].

The NA recognises (in clause NA.2.2.1) that buried structures with greater than 0.6m of fill and that are long (transversely) in relation to their span are effectively protected from climatic and operational temperature changes. The NA approach is based on BD 37/01[28].

In reality, the effect of cover must depend on the material. Some of the lighter fill materials (e.g. foamed concrete) are relatively good insulators. However, given the lack of data, it was considered justifiable not to distinguish between them.

NA.2.3 – Consideration of thermal actions [6.1.2(2)]

The BS EN 1991-5[20] offers two Approaches for dealing with the vertical temperature difference The first is a linear distribution (ΔT_{Mz}) which is based on an analysis of temperatures measured in a number of German bridges of different types of construction and different configurations of deck. The second is the incorporation of the non-linear terms (ΔT_{E}) that were derived by Emerson and her co-workers at the Transport and Road Research Laboratory (TRRL), later TRL. This latter method was based on a very comprehensive programme of theory and measurements on steel, composite and concrete bridges and formed the basis for the codified rules in BS 5400[22] and BD37/01[28]. The resulting temperature distributions from both methods were analyzed by Mirambell and Costa[17] and they concluded that the non-linear distribution as adopted in BS 5400 and now set out as Approach 2 in the Eurocode are suitable for representing the phenomenon of heat transfer in composite bridges realistically and that this approach allows for a more precise representation of the vertical temperature distribution that occurs through the deck.

NA.2.4 – Uniform temperature components – General [6.1.3.1(4)]

EN1991-1-5: **Figure 6.1** relates the maximum and minimum shade air temperature with corresponding maximum and minimum uniform bridge temperature respectively for the three types of bridge superstructure. The figure was based on meteorological data derived for the United Kingdom, and initially this was considered appropriate for all European Member States, subject to the need to extend the graphs on the figure to cover higher or lower shade air temperatures.

In the course of the drafting work on the Eurocode however it was found that the assumption of a single uniform temperature component corresponding to a particular maximum or minimum shade air temperature could be inappropriate, because the possible variations in daily shade air temperature within the Member States are too large. A means of allowing for this was developed but in the final document it was agreed to assume that the daily temperature range would be 10˚C. This provides values close to the empirical values adopted previously in the UK, in BD 37/01[28], although the match for steel box girder maximum uniform temperature components is not so good, suggesting that the BD37/01 uniform (effective) bridge temperatures may be too low. A brief note on the derivation of graphs for other daily ranges is given in the Background Document to ENV1991-2-5. A typical figure derived for steel boxes, for four daily/overnight ranges of shade temperature (0˚, 10˚, 20˚ and 30˚) is shown in figure 3 taken from the Background Document. The values adopted in BD37/01[28] are shown as circles (◦) in the figure.

The minimum and maximum uniform temperatures derived from these relationships take on an apparent degree of accuracy in EN1991-1-5: **Figure 6.1**. While the UK has confidence that they are accurate enough to be extrapolated to extreme conditions without resulting in any problematically incorrect bridge temperatures, they should not be regarded as ‘precise’.

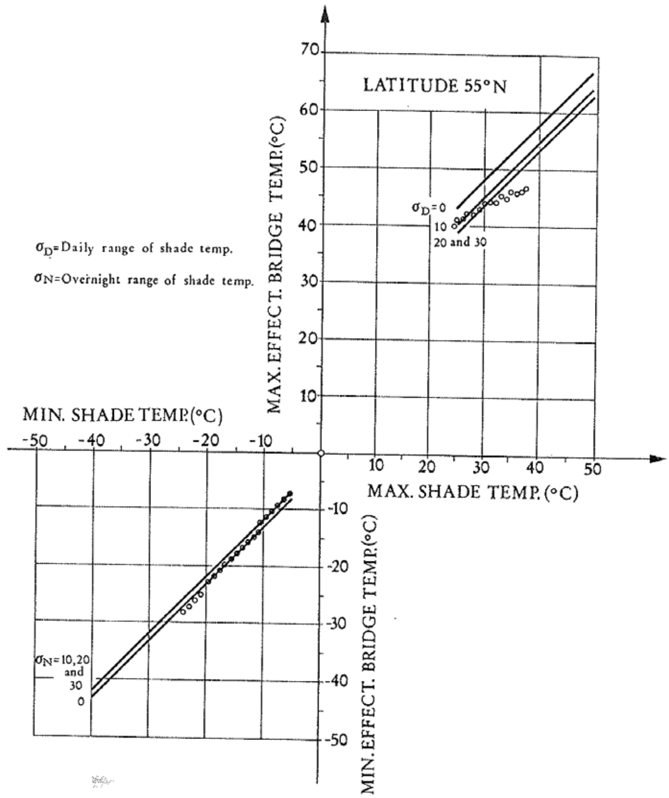

Figure 3. Relationship between max/min shade temperature and max/min effective bridge temperature for box girder bridges

It is important to note that EN 1991-1-5[20] makes no reference to adjusting the values for uniform temperature derived from Figure 6.1 to allow for a variation in surfacing depth. Appropriate adjustments are however given in the UK NA to BS EN 1991-1-5: **Table NA.1**, which replicates BD 37/01: **Table 12**. The data in UK NA to BS EN 1991-1-5: **Table NA.1** are based on the relationships presented in Figures 19 and 20 of TRL Report LR765[33]. It may also be seen that the slight scatter of the results associated with the different shapes of cross-section has been accommodated in Table NA.1 either by using mean values, or by rounding values up or down.

The procedure used to estimate the effect of depth of surfacing was based on work done by TRRL in the 1960s. Whilst there were some full scale measurements made by TRRL on steel bridges, there were none on composite and concrete structures. Thus the only way TRRL were able to investigate the effect of different depths of surfacing on a uniform bridge temperature was to carry out a theoretical exercise to calculate uniform temperatures under different depths of surfacing. Different deck cross-sections were also used. This work is described in Section 6 of LR765[33] and the results are shown in Figures 19 and 20 of that document.

The extreme minimum and maximum uniform bridge temperatures which were to be 'adjusted' by using these 'precise' theoretical corrections for surfacing depth had been derived from an assortment of relationships between minimum and maximum uniform temperatures and the shade temperature. These relationships and the methods used to establish them are fully described in LR744[16]. Emerson's concerns about the variety of assumptions made are set out throughout LR 744, and in Sections 6 and 7 of LR765[33].

The values adopted in the NA for adjustment of the uniform temperature component to allow for varying depths of surfacing were based on work undertaken over thirty years ago and adopted in BS 5400 Part 2[29] since 1978, and latterly in BD 37/01[28]. There is a need to revisit this work as the main author, Emerson, has doubts about its general application. Her research, out of necessity at the time, was extrapolated to enable Code clauses to be written. Likewise there was no means of researching this aspect further when writing the NA and the view was taken that the current procedures in use in the UK should continue to be accepted.

NA.2.5 – Shade air temperature [6.1.3.2(1)]

Thermal actions are an important and in some conditions governing source of effects to be considered during the design of bridges. The UK NA to BS EN 1991-1-5[20]contains maps of isotherms of extreme maximum and minimum shade air temperature within the UK (Figures NA.1 and NA.2) with a return period of 50 years. Recent sustained warming of the UK climate might be expected to have increased the severity of extreme high temperature events and reduced the severity of extreme low temperature events. During the preparation of the UK NA a limited exercise was commissioned by the Highways Agency to investigate whether the existing maps of isotherms of extreme shade air temperature contained in BD 37/01[28] that were developed in the 1970s, which present 120 year return period data collected over the period 1941-1970, would still be appropriate for current use.

Based on a limited selection of six UK stations, estimated *maximum* shade air temperatures corresponding to a return period of 50 years, based on data period 1975-2004, are typically approximately 0 to 1°C higher than those based on the 1941-1970 data. For one in 50-year *minimum* values the overall warming is rather more pronounced but the results are more scattered, with typical increases being approximately 0 to 2°C. It was therefore concluded that, for the purposes of BS EN 1991-1-5[20], maps produced using data from 1975-2004 would not be significantly different from the existing maps, which were based on a 50 year return period but adjusted to 120 year return period values for use in previous standards
UK mean shade air temperatures have recently (since about 1985) increased noticeably, from which it seems likely that the extreme value projections from the analyzed period (1975-2004) are already out of date with respect to the current decade's climate. Furthermore, an overwhelming consensus of climate change modelling predicts continued warming over coming decades. It was therefore suggested that an allowance should be made for any future trends by applying a factor to the extreme shade air temperatures. This factor would be reviewed every ten years; however this proposal was not accepted by the relevant BSI Committee.

NA.2.6 – Range of uniform bridge temperature component [6.1.3.3(3)]

The basis for the increase in the temperature range for bearings and expansion joints was the German Code DIN 1072[34] and the clause was drafted by the German group on the Eurocode Project team and specifically the Convener Professor G Konig. Supplement 1 to DIN 1072 explains that an allowance should be made to the temperature effects on bearings and expansion joints for uncertainties associated with moving and twisting of the supports and it quotes specifically: eccentricity of vertical loads, temperature differences in columns, supports and similar components, wind stresses, braking stresses, loads produced by various movement and deformation resistances of the supports and foundation.

NA.2.7 – Temperature difference components [6.1.4(3)]

During the construction of the in-situ joint between two balanced cantilevers, the precamber at the joint may be exaggerated as a result of the curvatures induced by the presence of a vertical temperature difference through the deck, potentially resulting in an angular discontinuity at the joint. The NA states that an initial temperature difference should be agreed on a project specific basis to make allowance for this effect. It would seem more reliable however to require the joint to be poured at a time of day when the temperature profile through the deck is stable (i.e. early morning or late evening).

NA.2.8 – Vertical linear component (Approach 1) [6.1.4.1(1)]

Approach 1 is based on the recommendations given within the German Standard DIN 1072[34]. It is recognised that usually the temperature profile is non-linear and unsteady and can be split into three components. Two of these components, the constant and the linear parts, are considered within DIN 1072[34]. The non-linear part which produces self-equilibrating stresses is however not addressed explicitly because it is considered that these effects are strongly dependent on the detailing of the cross-section. Thus the effect of the non-linear temperature distribution is considered by detailing rules within the relevant design codes (DIN 104[35]5 and DIN 4227[36]). It is likely that Approach 1 is based primarily on studies undertaken on concrete and composite bridge decks.

NA.2.9 – Vertical temperature components with non-linear effects (Approach 2) [6.1.4.2(1)]

The temperature difference profiles to be applied to the three different bridge deck types under Approach 2 are presented in BS EN 1991-1-5: 2003: **Figures 6.2 (a) to (c)**. They are based on the extensive research work undertaken by Emerson et. al. for TRRL during the 1970s and as such, reproduce the temperature difference rules given in BS 5400-2[29] and BD 37/01[28].

The basic profiles presented assume surfacing layer thicknesses of 40 mm for Type 1 and 100 mm for Types 2 and 3. Adjustments to the values of the temperatures within these profiles to allow for variations in the thickness of the surfacing layer are given within Annex B to the Eurocode (normative). Due to a drafting oversight, no reference is made to Annex B from within the body of BS EN 1991-1-5: 2003[20] itself, however the need to make such an adjustment is highlighted within clause NA.2.9 of the National Annex. Again, this approach provides continuity of the methodology previously used for the application of bridge effective temperatures in the UK.

The principles of the derivation of the constituent components of a temperature profile (ΔT_U, ΔT_{Mz} and ΔT_E) as presented in BS EN 1991-1-5: **Figure 4.1** of are outside the scope of this paper but are discussed in detail by Hambly[18].

The principal action of interest that results from the application of a vertical temperature difference profile through a section is the induced linearly varying component (ΔT_{Mz}), which induces a curvature in a structural element if it is unrestrained, or a moment if the section is restrained as a result of structural redundancy.

Note 2 to clause BS EN 1991-1-5:2003: **6.1.4.2** acknowledges that the small uniform temperature component (ΔT_U) derived from the application of the temperature difference is included within the uniform bridge temperature derived using clause BS EN 1991-1-5: 2003: **6.1.3** and as such, should not be applied as part of the temperature difference component.

Specific guidance on the consideration of the self equilibrating non-linear component (ΔT_E) is provided by other sections of the Eurocodes (e.g. EN 1992 etc.), where relevant.

NA.2.10 – Horizontal components [6.1.4.3(1)]

The action of the horizontal (transverse) temperature difference (ΔT_{My}) is not usually considered to be significant in bridge decks because the higher temperature on the edge exposed to solar radiation will have very low penetration as a proportion of the overall width of the deck and is therefore likely to have a fairly insignificant effect. In situations where the designer considers that the effect may be significant, the recommended value may be used or alternatively an appropriate profile may be derived from first principles.

NA.2.11 – Temperature difference components within walls of concrete box girders [6.1.4.4(1)]

There is currently very little UK specific data available on which to base firm recommendations in respect of temperature difference components within the webs of concrete box girders. The decision as to whether its effect will be significant enough to consider during design should be made on a project specific basis. As noted above, solar radiation will have very low penetration as a proportion of the overall width of the deck and is likely to be at relatively low levels of intensity at the times of day when webs are not protected by the shade of side cantilevers (morning or evening) and is therefore relatively unlikely to be significant except possibly in the case of deep boxes. The application of the recommended value will be conservative as it is of a similar order of magnitude to that applicable to a concrete bridge deck exposed to the peak intensity of solar radiation.

NA.2.12 – Simultaneity of uniform and temperature difference components [6.1.5(1)]

There are two key issues that lead to the treatment of the simultaneity of the maximum uniform and temperature differences being problematic in both practical and statistical terms. These are:

a) The statistical base of the two action components are different
b) During a typical daily heating/cooling regime, there will be a time lag between the attainment of the maximum values for each action component.

With respect to a), it has been possible to establish a reasonable relationship between the uniform temperature component in a bridge deck (previously termed bridge effective temperature) and the maxima and minima of a well recorded variable i.e. shade air temperature. However, no such clearly established relationship exists between the maximum positive and reverse temperature difference components and the variety of variables that influence their development (e.g. incoming solar radiation, re-radiation, material thermal conductivity and specific heat capacity). It should be noted that, as a result of this the temperature difference profiles given in BD37/01[28] are of magnitudes that might be expected to occur numerous times during a typical year, they are not extreme events.

If the time history of the development of temperature difference profiles through bridge decks is modelled and the uniform component and temperature difference components both plotted against time, it can be seen that there is a lag of several hours between the time at which the maximum temperature difference value occurs and the attainment of the maximum uniform component value, following absorption of heat from the top surface into the core of the deck cross-section. This lag will clearly vary with material type and will be smallest in the case of purely steel decks (which will have the most rapid thermal response and lowest thermal mass) and slowest for concrete (with a relatively slow thermal response and large thermal mass).

The simultaneity clause within BD 37/01[28] was based on a limited, observational study by Emerson and reported in TRL Report LR765[33]. The essence of the BD 37/01 provisions reflects the fact that the TRL observations suggested that:

i) The maximum positive temperature difference ($\Delta T_{M,heat}$ in Eurocode terms) could possibly occur simultaneously with bridge effective temperatures ranging between the maximum bridge effective temperature and limiting temperatures of 25°C in the case of steel decks and 15°C in the case of concrete and composite decks. It should be noted that these are observed events and not extreme values
ii) The maximum reverse temperature difference ($\Delta T_{M,cool}$ in Eurocode terms) could possibly occur simultaneously with bridge effective temperatures anywhere in the range between the minimum bridge effective temperature and effective bridge temperatures only slightly below the maximum bridge effective temperature. Again these comments relate to observed events and not extremes.

The provisions within the clause BS EN 1991-1-5: 2003: **6.1.5 (1)** were based on a proposal for the DIN standard, prepared by Sukhov but appear to be based on studies limited to concrete bridge decks. Their inclusion was strongly opposed by the UK delegates at the time.

Also unfortunately, during the drafting of the Eurocode a slightly unhelpful inconsistency has been introduced in terms of the symbols used. In section 4 (representation of actions) the terms uniform temperature component (ΔT_U) and linearly varying temperature component (ΔT_M) are introduced. Section 6.1.3 then defines the minimum and maximum uniform temperature components as $T_{e,min}$ and $T_{e,max}$ (rather than in terms of ΔT_U) to some extent mirroring the BD37/01[28] symbols for bridge effective temperature. These are then used in conjunction with the initial bridge temperature at the time of first restraint (ΔT_0) to establish contraction and expansion ranges ($\Delta T_{N,con}$ and $\Delta T_{N,exp}$) referred to in i) and ii)above.

BS EN 1991-1-5: 2003: **6.3 and 6.4** identify potential combinations of $\Delta T_{M,heat}$ with both $\Delta T_{N,con}$ and $\Delta T_{N,exp}$ as well as $\Delta T_{M,cool}$ with both $\Delta T_{N,con}$ and $\Delta T_{N,exp}$. However, the TRL observations demonstrated that only a subset of these permutations needs to be realistically considered in practice, as catered for within BD 37/01[28] and noted in i) and ii) above.

The intention of the UK NA to BS EN 1991-1-5: **NA.2.12** was to introduce the same simultaneity provisions previously included within BD37/01 by:

a) Reducing expressions 6.3 and 6.4 to a single, common relationship by setting both ω_N and ω_M to 1.0

b) Then giving guidance on how to adjust the range of uniform temperatures to be used in conjunction with the heating and cooling linearly varying temperature components.

Unfortunately the wording of the second bullet point of UK NA to BS EN 1991-1-5: **NA.2.12** doesn't make it sufficiently clear that the maximum reverse temperature difference could potentially occur in association with a uniform temperature component anywhere within the range from $T_{e,min}$ to temperatures only slightly below $T_{e,\,max}$ and some clarification will therefore be required.

In the light of the above, there is clearly scope to improve the guidance on the consideration of the simultaneity of uniform and temperature difference components in view of the limited data that have formed the basis of the current provisions; however this would require a reasonably significant programme of research. In the interim, it is considered reasonable to provide continuity of approach to thermal actions within the UK based on the country specific data for a variety of bridge forms, rather than adopting what are essentially unproven, general proposals based solely on research conducted on concrete structures.

To this end, it is likely that two methods for considering simultaneity of thermal actions will be made available:

i) The first taking account of the fact that the TRL observations clarify that maximum heating temperature differences cannot co-exist with the contraction range ($\Delta T_{N,con}$) but applying both the full expansion and full contraction ranges in all other case as a conservative simplification, as illustrated in Figure 4a.

ii) The second would then allow the simultaneity provisions included within BD 37/01[28] to be applied directly as originally intended, as illustrated in Figure 4b

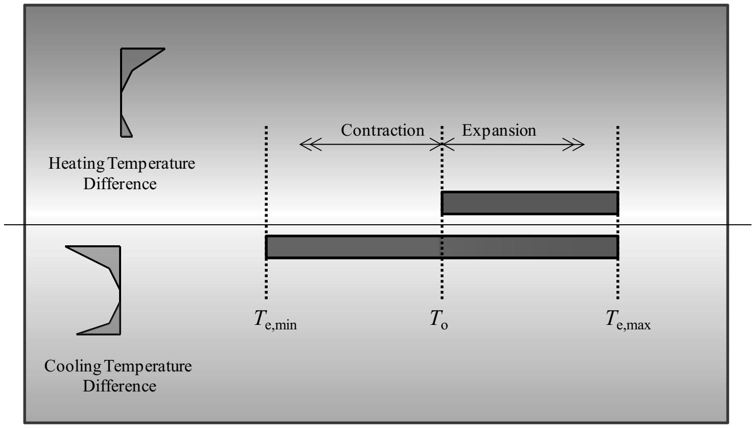

Figure 4a

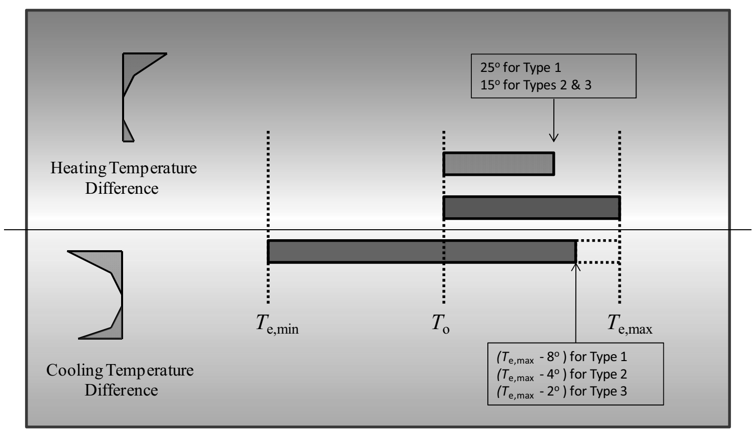

Figure 4b

NA.2.13 – Difference in the uniform temperature components between different structural elements [6.1.6(1)]

There is currently very little UK specific data available on which to base firm recommendations in respect of differences in the uniform temperature components between different structural components (for example a bridge deck and stay or suspension cables). The decision as to whether its effect will be significant enough to consider during design should be made on a project specific basis. The recommended values within the EN represent a reasonable starting point to establish the potential sensitivity of an individual structure. They are based on measurements made by Emerson on the Forth Bridge, reported in TRL Report

472[38] in 1999 and earlier studies by the Highways Agency on the towers of the Wye Bridge and piers of Aust Viaduct.

NA.2.14 – Consideration of thermal actions [6.2.1(1)P]

Because of the absence of any significant specific data on temperature differences through piers, the UK NA to BS EN 1991-1-5[24] advises the adoption of a simple equivalent linear difference profile, similar to Approach 1 for vertical profiles through decks. The most sensitive piers are likely to be those of modest height where a significant proportion of the pier is not shaded. In such cases the degree of restraint provided by the relatively high stiffness could lead to the development of reasonably significant bending effects. Where this approach is followed, recommended values are given in subsequent clauses.

NA.2.15 – Temperature differences [6.2.2(2)]

The recommended value for the equivalent linear temperature difference of 5°C seems reasonable and pragmatic. However, there would be merit in at least undertaking a sensitivity study to establish a degree of confidence in the proposed approach.

NA.2.16 – Temperature differences [6.2.2(2)]

Again the recommended value for the equivalent linear temperature difference of 15°C seems reasonable and pragmatic, although the value appears rather high for a wall, particularly in a retaining structure. Again there would be merit in at least undertaking a sensitivity study to establish a degree of confidence in the proposed approach.

NA.2.20 – Isotherms of national minimum shade air temperatures [A.1(1)]

The basis for the maps of isotherms presented in the UK NA to BS EN 1991-1-5: **Figures NA1 and NA2** is described in **NA2.5** above.

NA.2.21 – Isotherms of national minimum and maximum shade air temperatures [A.1(3)]

The recommended use of 10°C as the initial temperature T_0 is generally adopted in mainland Europe. The recommended value was based on data given in DIN 1072[34], and is referred to in the Background Document to the Draft Eurocode (ENV 1991-2-5).

In the UK, during the course of the TRL research, the annual mean shade air temperature was calculated from data from a small number of Meteorological Stations. The values ranged from 10.0°C to 10.6°C. The Meteorological Office considered that 10°C was a “good average working value for the annual mean shade temperature for most areas of the UK”. The annual mean uniform bridge temperatures for bridges in the vicinity of these Meteorological Stations varied between 10.8°C and 12.6°C. The bridges considered were concrete, composite and steel box.

According to the data to hand for the UK, the mean annual temperature for 1941 – 1970 for most of Scotland is between 8°C and 9°C, with some of the west coast lying between 9°C and 10°C. The mean annual temperature for 1941 – 1970 for most of England and Wales is between 9°C and 11°C. Of course the ‘mean annual’ temperature over 30 years is not identical to the ‘annual mean’ temperatures quoted, but they should not be very different.

However, whilst it may be reasonable to take a value of 10°C as an average uniform bridge temperature, for individual structures the actual uniform bridge temperature when they are constructed may be different. In cases where the temperature at which a bridge is restrained cannot be clearly defined and controlled, the NA therefore recommends that a range of values of T_0 be considered, and based on a pragmatic assessment of the possible minimum and maximum temperature recommends a cautious lower value of $T_0 = 0^oC$ be considered for expansion cases and a cautious higher value of $T_0 = 20^oC$ be considered for contraction cases. For buried structures a single value of $T_0 = 10^oC$ is however considered sufficient. The recommendations are similar to those in BD 31/01.

In drafting clause NA.2.21, it was not intended that the range of values of T_0 specified would necessarily be combined with the adjustments for bearing and expansion joints given in BS EN 1991-1-5: 2003 **6.1.3.3(3) Note 2.**

NA.2.22 – Maximum and minimum shade air temperature values with an annual probability of being exceeded p other than 0,02 [A.2(2)]

The coefficients k_1, k_2, k_3 and k_4 given in BS EN 1991-1-5: 2003[20] were adopted in the NA. These allow one to determine maximum and minimum shade air temperatures for annual probability of exceedences other than 0.02. They were derived by the Eurocode project team but agree closely with the method given in the paper by Hopkins and Whyte[37], which formed the basis of the extreme temperature maps in BD37/01[28] and the adjustments in the BD for reducing the 120 year return period values to 50 year return period values

NA.2.23 Temperature differences for various surfacing depths [B(1)]

There are several typographic errors in BS EN 1991-1-5: 2003[20] that are noted in this section of the NA[24]. The background to the adjustment for different surfacing depths is given in NA.2.9 above.

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BRIDGE DESIGN PROVISIONS OF UK NA FOR EN 1991-1-7 AND PD 6688-1-7

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Abstract
BS EN 1991-1-7 was published in 2006 to provide the requirements due to accidental actions on bridges. The UK National Annex to BS EN 1991-1-7 and the associated UK Publication Document PD 6688-1-7 were both published in 2008.
The preparations of UK National Annex and Publication Document were undertaken by Mouchel, Flint and Neill and Parsons Brinckerhoff for the Highways Agency and by RSSB. This paper describes the background to the development of the NA and PD that support BS EN 1991-1-7.

Introduction
This paper describes the background to the development of the UK National Annex to BS EN 1991-1-7:2006[2] and Published Document PD 6688-1-7:2009[3] relating to road bridges, footbridges, rail bridges and other road structures. It does not give background to BS EN 1991-1-7:2006[1] as this will be prepared by CEN (Comité Européen de Normalisation) in due course.

This paper covers various accidental actions on different types of structures. These are the accidental actions caused by road vehicles impacting with road bridges, footbridges and lightweight structures such as sign/signal gantries, lighting columns; by derailed rail traffic under or adjacent to structures; by ship traffic and by internal explosions inside tunnels.

Accidental Actions Caused by Road Vehicles – Impact on Supporting Substructures

Road bridges, including accommodation bridges
In the UK, impact loads on bridge supports have previously been designed to the requirements of BD 60[7]. In accordance with BD 60, **2.2**, a bridge support needs to be designed to withstand the vehicle collision loads if it is located at a horizontal distance less than 4.5m from the edge of the carriageway. If the bridge support is located at 4.5m or more from the edge of carriageway, the designer is required to analyse the vulnerability of the support to vehicular impact using engineering judgement and risk assessment. However it is doubtful whether a risk assessment is carried out in practice, possibly because there is no guidance given to analyse the vulnerability of the support to vehicular impact. In practice it is likely that designers will locate the support at 4.5m from the edge of carriageway in order to avoid designing the support to withstand the vehicle collision loads.

While the 4.5m rule is pragmatic and easy to apply, the basis of its derivation is unknown. It should be noted that in accordance with BS EN 1991-1-7, **Annex C**, it would not be possible to fully justify the 4.5m rule given in BD 60. On 28 February 2001 a vehicle came off the M62 motorway at Great Heck, near Selby, ran down the railway embankment and onto the East Coast Main Line, where it was struck by a passenger train. The passenger train was derailed and then struck by a freight train travelling in the opposite direction. Ten people on the trains were killed. The subsequent reports by the Health and Safety Commission[13], the Department for Transport[14] and the Highways Agency[15], form the basis for the development of guidance on the application of measures to manage risk where roads meet, cross or run close to railways.

Absolute safety cannot be attained at a cost that would be considered to be acceptable to society, given the competing needs for valuable resources. A risk-based approach to the allocation of the available resources does not therefore aim to meet an objective of absolute safety at any cost everywhere, but aims to ensure that finite resources are utilized so as to make the road system as safe as reasonably practicable for the largest number of people. It has been decided that, for the design for vehicular impact on bridge supports, risk assessment methods should be adopted in order to fall in line with the principles recommended in the Selby reports.

At the time of development of the risk assessment procedure the Highways Agency published its "Highway Economic Note No. 1", in which the cost and value of preventing a fatality was given as approximately £1.5m. For most highway bridges, the additional costs to society due to traffic delays will be considerably higher than this. Therefore judgements about the level of impact protection for highway bridges are usually governed by economic evaluations of traffic delay costs and benefits rather than by the potential danger to life. The historical record indicates that the risk to human life from the collapse of road bridges following accidental impact, is too low to justify any particular action to mitigate such risks. However, the economic argument is much more significant, and it was found that special protection measures were justifiable where the combination of risk and consequence at a particular location was more than 2.4 times the average historical value. The risk ranking factors are designed to require higher level of safety provisions where this risk level is exceeded.

In the case of footbridges, the risk to life could be much higher, and the risk assessment parameters are weighted accordingly. The design values for impact loads on supports adjacent to motorways, trunk and principal roads in the UK NA to BS EN 1991-1-7, are generally based on the BD 60 requirements. In the Eurocodes, the design values of accidental actions are given directly, and there are no additional safety factors applied (see the UK NA to BS EN 1991-1-7, **Introduction**). For this reason, the nominal loads in BD 60 should be multiplied by 1.65, which is a product of the partial safety factor γ_{fL} of 1.5 and the γ_{f3} factor of 1.1, to obtain the design values. Therefore, the equivalent static design forces F_{dx} in the UK NA to BS EN 1991-1-7, **Table NA.1** for motorways for example, should be equal to 1650 kN, based on the value of 1000 kN in BD 60, **Table 3** multiplied by 1.65. It should be pointed out that the current design values in the UK NA to BS EN 1991-1-7, **Table NA.1**, do not include the γ_{f3} factor of 1.1. In order to be equivalent to previous practice in BD 60, the values in the UK NA to BS EN 1991-1-7, **Table NA.1**, therefore need to be multiplied by 1.1. An amendment to the UK NA to include the γ_{f3} factor is currently being considered.

Depending on the risk ranking factor R_{de}, as defined in the UK NA to BS EN 1991-1-7, **NA.2.11.2.3.3**, an adjustment factor Fa is determined based on the threshold values of T_a and T_b. The risks are categorised into high, normal and low for which adjustment factors of 2, 1 and 0.5 are applied respectively. The adjustment factor is intended to be a factor which is applied to the design loads given in the NA to BS EN 1991-1-7, **Table NA.1**, except for the minimum robustness requirement.

The default threshold values for T_a and T_b, are given as 2.4 and 0.5 respectively in the UK NA to BS EN 1991-1-7, **NA.2.11.2.4**. These values are based on a calibration study using the bridges in Highways Agency Area 11 (Warwickshire, Leicestershire and Staffordshire), and are intended to provide the level of safety required without a disproportionate number of structures falling into the high risk category. It should be noted that the UK NA to BS EN 1991-1-7, **NA.2.11.2.4**, requires that these threshold values are used “unless otherwise specified for the individual project”. This provides a mechanism for different threshold values to be used by other highway authorities to suit the requirements of their particular bridge stock. The adjustment factor of 2 defined in the UK NA to BS EN 1991-1-7, **NA.2.11.2.4.2**, for high risk bridge supports, is based on the full scale dynamic testing carried out by TRL and Ove Arup. This testing report[16] suggested a factor of 2.8 to be applied to the design values in BD 60. However, it was considered that this would be extremely conservative, with all the unfavourable factors occurring simultaneously.

The dynamic factor of 2 is a classical solution, derived from the behaviour of a weight, sitting on top of an unloaded span, which is suddenly released. The dynamic effect of the sudden load takes the mass through the static equilibrium point and then a similar distance beyond. The stored energy of the span, due to its bending stiffness, then returns the mass back through the equilibrium point to the starting point, with the cycle then repeating. In an undamped scenario, this is classic simple harmonic motion. The measured collision loads might be argued to require a dynamic factor of only one, since they were measured during actual testing and therefore represent all of the dynamic interactions. The factor might even be argued as less than one, since the forces are of such short duration that the mass of the pier might not have time to accelerate and hence deflect. This is an important consideration when attempting to rationalise the static equivalent loads used in design. However, it was decided that the factor of 2 was appropriate as a conservative upper bound.

In the design, it is best to achieve a lower risk ranking factor (R_{de}) so that the risk is reduced from high to normal. If this is not practicable, the preferred option is to provide safety barriers in accordance with PD 6688-1-7, **2.7**.

The adjustment factor of 0.5 in the UK NA to BS EN 1991-1-7, **NA.2.11.2.4.4** for low risk supporting structures, is based on engineering judgement. As an alternative option to designing a support for the impact values in the UK NA to BS EN 1991-1-7, Table NA.1, a higher containment barrier may be used in accordance with PD 6688-1-7, **2.7**.

It is rare for a Heavy Goods Vehicle (HGV) to collide with a bridge support. It is even more rare for there to be significant structural damage. Collisions that cause a bridge closure or structural collapse are rarer still. It was therefore difficult to directly calculate the actual risks. Only one bridge collapse was found in records of the past 40 years, and that was for a bridge design that was particularly vulnerable to impact from vehicular collision. Since this was not a good statistical sample, it was assumed that this event was representative of low consequence events for a series with a typically one in ten year collapse frequency.

Historically, due to the lack of statistical data, some risk ranking systems have chosen arbitrary scales for the individual risk contributions, typically one to five or one to ten. However, it was considered to be more reasonable to base the scales on engineering judgement, with justifications, rather than choosing an arbitrary scale: just because we cannot rigorously assign risk factors does not mean that we should not at least provide a best estimate which can be revised in the future in the light of experience and improved knowledge. Some factors, such as road type, might have a small effect, with a range of factors around five, but other factors, like the number of HGV's, might be vastly different between a country lane and a major motorway for example.

The risk assessment method developed, calculates an average risk, and utilises factors that modify this average risk. Its value depends on the influence of the factors described in the following.

F_1 – Road class below bridge

Accident rates for different road types are published each year, giving accident rates per vehicle kilometre. This factor was based on these accident rates assuming that the rates of HGV collisions with piers follow the general trend of road accidents.

F_2 – Factor for HGV flow under bridge

Although the accident rates may be non-linear, with medium traffic flows causing proportionally more vehicle conflict than low flows, and high flows sometimes causing slow moving traffic jams, it seemed reasonable to equate collision risk to the number of HGV's passing the site under consideration.

F_3 – Influence of speed limit under bridge

Intuitively, it appears logical that the forces will increase with the square of the speed and that the speed factors should therefore be greater than the values in the UK NA to BS EN 1991-1-7, **Table NA.4**. However, the historical impact tests showed that the resultant force on the structure, depended on how strong certain elements of the vehicle were (e.g. engine bolts shearing off at the first 5000 kN peak). Furthermore, it was evident that the energy involved is more likely to determine how long the forces are applied for, rather than determining the actual force. This has some effect on the equivalent static force applied to the structure but it is not a direct link. The speed limit factors are therefore somewhat arbitrary, and assume that vehicles travelling at higher speeds have less time to react to problems, thereby increasing the risk of an impact.

F_4 - Influence of junctions

This is a set of factors that requires judgement. The factor was included because more sudden weaving manoeuvres happen near junctions and hence the accident risk is higher.

F_5 – Influence of clearance

Historically there was an all or nothing decision to apply the full collision load based on a cut off of 4.5m from the edge of the carriageway. The definition of carriageway included hard shoulders on motorways since lorries would use these during highway maintenance. The clearance factors used in the UK NA to BS EN 1991-1-7, **Table NA.6** were, however, based on the distance from the centreline of the nearest lane, since that was the starting point for most incidents. Hard shoulder is rarely used, although it may be necessary to use judgement where hard shoulder running in peak times is applied routinely.

The 4.5m "Cliff edge" cut off is no longer applied and a system based on graduated clearance from the centreline of the nearest running lane for HGVs, is used instead. This was based on a Monte Carlo analysis, with a large sample of possible incidents using controlling values of initial speed, initial departure angle, deceleration and cornering ability. The initial speed and departure angles, were based on average values of 55mph and 20^{o}, with the assumed lorry handling characteristics based on discussions with an experienced lorry driver.

The analysis showed that lorries can run a long way off of the carriageway, even if the driver is trying to recover from an incident.

$\mathbf{F_6}$ – Number of columns for each support type and $\mathbf{F_7}$ - Factor for stability of deck

These values were chosen based on engineering judgement, with a range of values proposed by contributors to development of the risk assessment process.

It should be noted that curvature of the road was, counter-intuitively, not taken into account in the F factors because a study[17] indicated that accident rates are higher on straight roads than on curves.

$\mathbf{F_8}$ – Consequence factor for road bridges

This factor takes the risk to life into account. The number of lives that might be lost are related to the number of vehicles, and hence road users, that might be crushed under, or crash into the side of, a collapsing bridge, as well as those who might lose their lives whilst travelling over a collapsing bridge.
Factor $\mathbf{F_8}$ contains three terms:

i) A constant term, that is proportional to the average justifiable spend to reduce fatal accidents
ii) A variable term that is proportional to the cost of interrupting the flow of vehicles passing under the bridge
iii) A variable term that is proportional to the cost of interrupting the flow of vehicles passing over the bridge

Where flows below and on the bridge are similar, the cost of interrupting flow is assumed to be much higher for over-bridge traffic, since under-bridge traffic ought to be able to flow freely after the wreckage has been cleared away, whilst over-bridge traffic must wait for a new or temporary bridge. Since the formulation of F_8 only varies with traffic flow, it cannot be applied to footbridges. In order to achieve a minimum level of robustness to resist impact forces for all bridges, minimum values for the design forces are specified. BS EN 1991-1-7, **4.1(1) Note 3**, permits guidance to be given for the transmission of impact forces to foundations as non-contradictory complementary information (NCCI). The UK NA to BS EN 1991-1-7, **NA.2.10** refers to PD 6688-1-7, **2.6**, for recommendations on design of the foundations for impact. Resistance to the impact forces from the ground should be established for design using BS EN 1997-1 with the following qualifications:

i. Only ULS checks are required
ii. when checking against the sliding of the base and bearing capacity, the collision loads should be reduced by 50% and,
iii. full loading should be considered for checking against overturning.

Lightweight bridges such as foot and cycle track bridges

In accordance with BS EN 1991-1-7, **4.1(1) Note 1**, the requirements for accidental actions on lightweight bridges, such as foot and cycle bridges, can only be referenced in the UK NA to BS EN 1991-1-7 as NCCI. The UK NA to BS EN 1991-1-7, **NA.2.9**, refers to PD 6688-1-7 for this information. PD 6688-1-7, **2.5.1**, provides recommendations for the design impact forces to be used for foot and cycle track bridges.

It is assumed that footbridges are intrinsically lightweight and the risk assessment is undertaken to determine the adjustment factor (F_a) to be applied to the equivalent static design forces in PD 6688-1-7, **Table 1** and, where appropriate, the level of containment of the barrier to be used to protect the footbridge support.

The risk assessment method in the UK NA to BS EN 1991-1-7 has been adopted for footbridges. However it is extremely sensitive to factor F_8 which is given in PD 6688-1-7, **Table 2**.

Consequence for foot/cycle track bridges - Factor F_8

The F_8 value for foot/cycle track bridges, is used to ensure that the available resources are allocated in proportion to the potential value to society in allocating resources where they can achieve maximum benefit. A bridge which carries very few pedestrians does not merit the same degree of protection as one that is very heavily used by pedestrians/cyclists or which carries occasional large crowds (sports events for example).

i. The standard value "1.0" applies to bridges in urban areas where, typically, a bridge might well be most likely to be struck when roads and footways are busy and a typical bridge might be expected to be carrying somewhere in the region of 40 persons at any one time.
ii. "Sub-urban" bridges are assumed to be likely to carry about half as many people at any one time as their urban counterparts, so they have a factor of 0.5.
iii. Rural bridges are assumed to carry typically only one tenth as many persons, and a factor of 0.1 is applied.
iv. Heavily used crossings, where dense crowds are likely to congregate, are treated as special cases, and a factor of 5 is applied in these circumstances, in order to maximize the likelihood that protection will be provided.

Lightweight structures such as sign/signal gantries

The requirements for design of sign/signal gantry supports to resist collision forces are given in BD 51[6] and TD 19[9]. No further information is given in PD 6688-1-7.
However, it is envisaged that a risk assessment method will be developed in future for the design of sign/signal gantry supports to resist collision forces. As walkway access is unlikely

to be provided for new gantries, the consequence factor F_8 and the level of containment of barriers required, should be much less onerous than those specified for foot/cycle track bridges.

Lightweight structures such as minor structures described in BD 94; e.g., lighting columns, traffic sign/signal posts

No information is given in PD 6688-1-7 for design for collision forces to be applied to the supports of minor structures. It is envisaged that the requirements will be based on existing standards such as BD 94[8], TD 19[9] and TD 89[11].

Accidental Actions Caused by Road Vehicles – Impact on Supporting Superstructures

The design values for impact loads on superstructures above motorways, trunk and principal roads in the UK NA to BS EN 1991-1-7, **NA.2.16**, are generally based on BD 60[7]. As for the loads on supports, there is no additional safety factor for accidental actions and the design values are given directly. For this reason, the design loads should be based on the nominal loads in BD 60 multiplied by 1.65, which is a product of the partial safety factor γ_{fL} of 1.5 and the γ_{f3} factor of 1.1. Therefore, the equivalent static design forces F_{dx} (825 kN) and F_{dy} (415 kN) in the UK NA to BS EN 1991-1-7, **Table NA.9** for motorways, are derived from the BD 60, **Table 4** values, multiplied by 1.65.

For other rural and urban roads, the equivalent static design forces given in the UK NA to BS EN 1991-1-7, **Tables NA.9** and **NA.10** are based on the values that are pro-rata to those given in BS EN 1991-1-7, **Table 4.2**, except for courtyards and parking garages.
The threshold value of the headroom clearance h_1, above which there is no need to design for impact on superstructures, in the UK NA to BS EN 1991-1-7, **NA.2.17**, is taken as 5.7m which is the same as BD 60, **Table 2**.

The vertical sag curve compensation values given in PD 6688-1-7, **Table 3** are based on the values in TD 27, **Table 6-1**.

The recommendations for provision of restraint to the deck of foot/cycle track bridges in PD 6688-1-7, **2.5.2**, are similar to the recommendations in BD 60, **2.2**. However, as these impact forces are applicable where the headroom is h_1 and above, they need only be designed for within the deck restraint and the forces need not be designed for in the supporting column and foundation. The aim is to ensure there is adequate restraint on the deck in order to prevent the deck being removed from the support when subjected to these impact forces.

Accidental Actions Caused by Derailed Rail Traffic Under or Adjacent to Structures

Background

The current requirements for the design of railway structures to resist derailment actions, are set out in Railway Group Standard GC/RT5112, Issue 2[18], **Part 7**. This document superceded the following standards and sources of guidance:

GC/RT5112, Issue 1[19]

GC/RC5510[20]
GC/TT0112[21]

GC/RT5112, **Part 7**, allows for the situation that existed prior to the introduction of the Eurocodes, where designs were undertaken to BS 5400-2[22], and also the current situation, where railway bridges are required to be designed to the Eurocodes. The Eurocode design requirements for derailment and other actions are set out in BS EN 1991-2, **6.7**. The scope of BS EN 1991-2, **6.7** is limited to derailment actions that impose loads on the bridge deck (see BS EN 1991-2, **6.7.1**). The requirements for actions due to derailment under or adjacent to a structure, are stated in BS EN 1991-2, **6.7.2(1)**. This clause requires the actions due to collision following a derailment, to be based on the requirements of BS EN 1991-1-7 and, where relevant, its National Annex.

Types of rail traffic

The types of rail traffic for which the design rules in the UK NA to BS EN 1991-1-7 are applicable, are set out in UIC 777-2R[23]. The recommendations are linked to the class of structure, type of traffic and line speed, as shown in Table 1:

Class of structure	Type of traffic	Line speed (km/h)
A	Mixed traffic	120
B	Passenger	300
B	Freight	160

Table 1. UIC Rail traffic types

Formerly, BS 5400-2 did not provide information for the design loads to be used for impact with the supports and superstructures of bridges over railways. The design requirements for supports are contained within GC/RT5112, although the design forces given for supports are not explicitly linked to the class of structure or the line speed. The design impact forces are deemed to be appropriate for mixed traffic and it is generally assumed that, for non-high speed lines, the limit of validity for the impact forces in GC/RT5112, **Part 7**, is based on impacts from conventional rail traffic travelling at a maximum possible line speed of 200 km/h.

For line speeds greater than 200 km/h (appropriate to passenger trains only), no provisions are made in GC/RT5112, **Part 7** but the possibility is recognised in UIC 777-2R for Class B structures.

Classification of structures

BS EN 1991-1-7, **Table 4.3**, provides a classification for structures that may be subject to impact from derailed traffic. The classification is based on structures that *'span across or near to the operational railway'* that have the characteristics set out in Table 2:

Class of Structure	Characteristics
A	i. Buildings that are permanently occupied. ii. Buildings that serve as a temporary gathering place for people. iii. Buildings that consist of more than one storey.
B	i. Bridges carrying vehicular traffic. ii. Single storey buildings that are not permanently occupied. iii. Single storey buildings that do not serve as a temporary gathering place for people.

Table 2. Classification of structures

Although bridges are excluded from being categorised as Class A structures according to the BS EN 1991-1-7 classification, it is possible that certain heavily trafficked bridges over busy railway lines, could be classified as Class A structures. The UK NA to BS EN 1991-1-7, **NA.2.23**, allows for this possibility.

Design values of impact force

The design values for impact force on Class A structures are stated in BS EN 1991-1-7, **4.5.1.4 (1)**, as being relevant for rail traffic with a maximum speed of 120 km/h. The UK NA to BS EN 1991-1-7, **NA.2.25**, accepts the recommended values in BS EN 1991-1-7, **4.5.1.4(1)** for Class A structures. These values are greater than the design impact forces in GC/RT5112, **Part 7**, but are intended to be relevant where there is a high risk to people in buildings where the supports are vulnerable to impact. The design impact forces for Class A structures from BS EN 1991-1-7, **Table 4.4,** are summarised in Table 3 below:

Distance "d" from structural elements to the centreline of the nearest track (m)	Force F_{dx}[a] (kN)	Force F_{dy}[a] (kN)
Structural elements: d < 3m	To be specified for the individual project. Further information is set out in Annex B.	To be specified for the individual project. Further information is set out in Annex B.
For continuous walls and wall type structures: 3m < d < 5m	4000	1500
d < 5m	0	0
a – x = track direction; y = perpendicular to track direction		

Table 3. Design impact forces for Class A structures

It may be inferred from the values quoted and the associated traffic speeds (see Table 1) that there is a direct link between the design impact forces according to train type and speed and distance from the track. However, the nature of derailments is very uncertain and the way in which a train behaves following a derailment, is likely to be influenced by a number of factors. Guidance on the factors to be taken into account is included in BS EN 1991-1-7 **Annex B**. This guidance is based on that contained in UIC leaflet 777-2R, which contains additional recommendations and guidance to that in BS EN 1991-1-7 **Annex B**. This guidance is intended to be used in conjunction with a risk assessment undertaken to establish the likelihood and consequences of a derailment close to the structure supports.

The intentions in the UK NA to BS EN 1991-1-7 **NA.2.30**, are that a minimum level of robustness is provided for Class B structures, such as bridge supports, located within the hazard zone and also that collapse of the bridge deck onto passing rail traffic is prevented. This is achieved by designing the supports to sustain light impacts from derailed traffic and the deck structure to continue to function under the influence of reduced traffic load on the bridge over the track(s). These design requirements will achieve the design objectives to maintain safety of the railway, the safety of users of overbridge structures and to minimise the disruption to rail and road traffic or pedestrian access, in the event of an impact following derailment. In this way, it does not matter if the impact force is actually greater than the design force and, the design of structures to resist impact can be achieved at a reasonable cost.

Where design solutions involve the provision of resistance to impact, it may be helpful to consider what the selected design forces might mean in terms of the design situation that they represent. The design impact force is a function of the mass of the train acting on the structure, the speed at impact, and the stiffness of the structure and the train. For a typical ten car passenger train, the total train weight might be 400 tonnes (10 x 40 tonnes) and the line speed could be up to 200 km/h (125 mph). In a head-on collision, it is possible, but unlikely, that the full weight of the train will be taken by the structure, as individual vehicles may become detached or continue to be resisted by the track. The particular circumstances of a derailment situation are likely to be complex and will depend on, for example, the train speed and track geometry. If it is assumed in a particular derailment situation, that the weight of the train impacting the structure is 80 tonnes and also that the soft impact conditions described in BS EN 1991-1-7, **C.2.2** are relevant, the required impact resistance may be determined from the assumption that the kinetic energy of the train is arrested by the work done in deforming the structure.

Expression C.5 from BS EN 1991-1-7, **Annex C**, is repeated below:

$$\tfrac{1}{2}mv_r^2 = F_0y_0$$

$$\text{and } F_0 = mv_r^2/2y_0 \qquad \text{Equation 1}$$

Where:

F_0 = Impact force that the structure can resist to coincide with the limit of its plastic strength
y_0 = The limit of displacement of the structure in achieving its plastic strength
m = mass of the train resisted by the structure v_r = velocity of the train at impact

For a reasonably flexible structure with a displacement of say 100 mm, the impact forces range from 77 MN for impact at 50 km/h to 1235 MN for impact at 200 km/h. For a structure of similar flexibility (100 mm assumed) and a reduced proportion of the train mass acting on the structure, the impact forces range from 4 MN for impact at 50 km/h to 62 MN for impact at 200 km/h. This demonstrates the following characteristics of dynamic impacts:

- The impact force is significantly influenced by the energy dissipated during impact (in this case by a factor of approximately 20 and pro-rata to the reduction in train mass acting on the structure)
- Impact forces can be very large for only a proportion of the full train weight impacting at moderate speeds
- An impact force of 4 MN, which is appropriate for design of Class A structures, might be achieved for impacts from a mass as little as 4 tonnes at a speed of 50 km/h.

Forces of very large magnitude were clearly involved in the Eschede train disaster, where a 12 car ICE train travelling at a speed of 200 km/h between Hanover and Hamburg in Germany on 3 June 1998, demolished the supports of the bridge over the track, causing the bridge to collapse. Despite the very great number of fatalities and severe injuries (101, 88) out of a total of 287 passengers, the likelihood of such events is mercifully very low. In practice, designing structure supports to resist such enormous forces cannot be economically achieved or justified.

Designing the structure itself to resist impact forces is only one solution that may be employed to achieve the design objectives stated previously. Other solutions that might be employed are:

i. Preventing impact with a structure (protection provided to the structure or preventing the train from leaving the track for example).
ii. Reducing the likelihood of impact with a structure to a tolerable level (locating switches and crossings away from structure, reducing speed and increasing clearance, for example).

Clearly, options involving all or part of the solutions described above, are also possible as long as the risks are considered to be controlled to an extent that is sufficient.

Structures located in areas beyond track ends

Situations where rail traffic may overrun the end of the track (for example at terminal stations), may give rise to high risk situations, particularly where buildings are located in areas beyond the track ends. The limit of the hazard zone, beyond which it is not considered to be necessary to design for impact from overrunning trains, is specified in the UK NA to BS EN 1991-1-7, **NA.2.31**.

Where it is necessary to locate structures within the hazard zone, BS EN 1991-1-7, **4.5.2(4)**, recommends that an impact wall be provided to prevent the train impacting the structure.

Accidental Actions Caused by Ship Traffic

The UK NA to BS EN 1991-1-7, **NA.2.34**, **NA.2.37**, **NA.2.40** and **NA.2.41**, does not provide alternative design values to those recommended in BS EN 1991-1-7. It is recommended that the design requirements should be specified for the individual project. This is consistent with previous practice, where the design requirements were dependent on the specified shipping movements below the proposed bridge, in consultation with those responsible for the navigable waterway concerned. However, advice on the design requirements for 'immersed tube tunnels' subject to loads from a 'sunken/grounding ship' or 'falling/dragging anchors', is given below.

Immersed tube tunnels

Immersed tube tunnels are frequently selected as the most appropriate design solution for crossings beneath navigable waterways. Typically, immersed tube tunnels are placed below the river / sea bed, with a rock or other protection layer over the roof of the tunnel and the adjacent backfill, to protect the tunnel from local damage and the effects of scour.

The marine conditions can vary considerably from case to case. The depth of water over the tunnel, and the profile of marine traffic, have a significant bearing on the magnitude of potential accidental actions due to impact from a sunken/grounding ship or a falling/dragging anchor.

The likelihood and magnitude of these actions are likely to be site specific and they should therefore be established on an individual project basis, with reference to data provided by the local harbour master, or other similar marine authority. The characteristics of vessels using the waterway should be taken into consideration (for example, length, beam, hull depth, propeller type and deadweight), as should the frequency and nature of the cargo transported.

Sunken/grounding ship

The depth of water over the tunnel may be sufficient for large ships to sink and be completely immersed. However, the under keel clearance of immersed tube tunnels, is frequently just sufficient to allow clearance for ships with the deepest draft. Consequently, in shallow water, the case of grounding, or stranding, may be more appropriate.

Typically, immersed tube tunnels are designed for a uniformly distributed static loading of $50kN/m^2$ applied to a 30m length of tunnel across the full width, but the magnitude and pattern of loading will depend on the ship(s) considered in design, and magnitudes up to $200kN/m^2$ are possible. An equivalent static concentrated load may be derived to represent the impact from a sunken/grounded ship.

Ship loading should be considered in the design of the structure, the provision of watertight joints and, the assessment of settlements, taking into account the time taken to salvage the sunken/grounded vessel.

Falling/dragging anchor

The effects of an anchor impacting the tunnel structure directly, or being dragged across the line of the tunnel structure, should be considered. Either the tunnel structure should be designed to resist the full loading imposed by the design anchor, or the tunnel protection should be designed to mitigate the effects. The design anchor should be appropriate to the shipping using, or expected to use, the waterway.

The penetration of a falling anchor in a concrete or rock layer, is normally estimated using the report ‘Concrete Structures Under Impact and Impulsive Loading’, Synthesis Report, Bulletin d'Information No. 187, Comité Euro-International du Beton (CEB), 1988. An equivalent static load is applied to the tunnel roof.

If designed to mitigate the effects of a dragging anchor, the tunnel protection layer should be sufficient in depth and extent either side of the tunnel, to ensure that anchors break free from the protection layer before snagging the tunnel structure.

Internal Explosions in Road and Rail Tunnels

The UK NA to BS EN 1991-1-7:2006, **NA.2.42**, specifies that internal explosions in road and rail tunnels should be specified for individual project. The NA recommends consideration of the method set out in BS EN 1991-1-7, **D.3** for determination of design pressures in road and rail tunnels. However, this may not be appropriate in all cases and it is possible that clients may wish to define the design situations for explosions in road and rail tunnels. It is further recommended that specialist advice be sought where necessary.

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PROVISIONS OF THE UK NA FOR EN 1991-2 FOR HIGHWAY AND FOOTBRIDGES

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Abstract
BS EN 1991-2 was published in 2003 to provide the traffic load on bridges. The UK National Annex (NA) to BS EN 1991-2 was published in 2008 and a UK Published Document is under preparation. The paper only covers highway bridges and footbridges. Rail bridges will be covered in a separate paper.
The works on the preparation of UK National Annex and Publication Document were undertaken by Atkins, TRL, Flint and Neill, Parsons Brinckerhoff and Arup for the Highways Agency. The paper describes the background to the development of the NA to BS EN 1991-2.

Introduction
This paper describes the background to the development of the UK National Annex to BS EN 1991-1-2[1] and Published Document PD 6688-2:2010[2] relating to bridges and other highway and rail structures. It does not give background to BS EN 1991-2:2003[3] as this will be published in due course.

This paper covers the background for traffic load models, horizontal loads, fatigue load models, impact loads, forces due to collision with vehicle restraint systems, abutments and wingwalls adjacent to bridge, pedestrian loading, and footbridge vibration and associated acceptance criteria.

Traffic load models

Load model 1 (LM1)
Load Model 1 (LM1) given in BS EN 1991-1-2, **4.3.2** is intended to represent the global and local effects of normal traffic that includes cars, light goods vehicles and rigid and articulated heavy goods vehicles up to a gross weight of 44 tonnes. These vehicles are governed by the EC Directive [4] and implemented through the C&U [5] and Authorised Weight [6] Regulations.

LM1 comprises a double-axle concentrated loads tandem system (TS) superimposed over a uniformly distributed load (UDL), the intensity of which remains constant with loaded length. Type HA loading model given in BS 5400-2:2006 [7], **6.2** and BD 37 [8] has been used satisfactorily for many years in the design of bridges in the UK. Type HA loading model comprises a UDL, the intensity of which varies with loaded length and a constant Knife Edge Load (KEL) of 120 kN. Despite the clear differences between the two load models, based on the comparison of extreme static load effects from different sources, it was considered that Type HA loading model could be a reasonable basis for the calibration of load adjustment factors (α) for LM1.

The load adjustment factors (α) for LM1 are given in the NA to BS EN 1991-1-2, **Table NA.1**. These load adjustment factors were developed after a calibration study which took into account the differences between the two load models and hence various factors that influence the calibration. These are i) divisions of the carriageway into notional lanes and notional lane width; ii) load intensities of UDL, TS and KEL and iii) lane distribution of loading and lane factors. Further information is given in background reports [9] [10].

Load model 2 (LM2)

Load Model 2 (LM2) is used to determine the local effect of very short loaded length, where LM1 is inappropriate. The adjustment factor α for LM2 is given in NA to BS EN 1991-1-2, **NA.2.14**. However it should be noted that LM2 loading appears to be much greater than the wheel loads given in BS 5400-2:2006, **6.2.5**. Consideration might be given to apply a lower β factor to, say 0.8 which is the value given in the French National Annex.

For LM1 and LM2, the vehicle may use super single tyres or twin tyres. For simplicity, the more onerous wheel contact surface for LM1 is also adopted for LM2, see NA to BS EN 1991-1-2, **NA.2.15**.

Load model 3 (LM3)

Load Model 3 (LM3) given in the NA to BS EN 1991-1-2, **NA.2.16** represents the effects of special vehicles that do not comply with the C&U and Authorised Weight Regulations. Unlike LM1, the vehicles are not governed by an EC Directive. Instead they are governed by the STGO Regulations [11] for vehicles of gross weight or trailer gross weight of 150 tonnes or less and Section 44 of the 1988 Road Traffic Act [12] for vehicles of gross weight or trailer gross weight of over 150 tonnes.

The models of special vehicles given in BS EN 1991-1-2, **Annex A** have not been adopted as the load models given in BD 86 [13] are considered a more appropriate basis as they were developed specifically for the special vehicles in the UK.

Load model 3: SV load model (LM3)

The SV load models are given in the NA to BS EN 1991-1-2, **NA.2.16.1** and are similar to those given in BD 86. The SV load models are essentially adopted and thus replace the Type HB loading model given in BS 5400-2:2006, **6.3** as they give better estimation of load effects of real STGO vehicles. This approach has the added advantage that the load models for design and assessment are the same. Further information is given in the background report [14] on BD 86.

Load model 3: SOV load model (LM3)

The SOV load models are given in the NA to BS EN 1991-1-2, **NA.2.16.2** and are developed based on the details of Special Order (SO) vehicles contained in the Highways Agency's AIL

(Abnormal Indivisible Load) database over the period from 2000 to 2005. The SOV load models are intended to closely represent the load effects of real SO vehicles.

Further information is given in the background report on the characteristics of Special Order vehicles[15] and background report on the loading models for Special Order vehicles[16].

Horizontal loads

The horizontal forces associated with LM3 were based on a research carried out by TRL[17].

Fatigue load models

The five fatigue load models modified in the NA to BS EN 1991-1-7:2006 [18], **NA.2.22 to NA.2.27** are based on BS 5400: Part 10: 1980 [19] and previously developed fatigue load models for the Highways Agency. The objective of the modifications in the NA is to obtain similar design lives to those obtained using BS5400: Part 10: 1980 for similar design details used in similar circumstances and to take the opportunity to up-date the vehicle fatigue spectrum.

Further information is given in the background report [20] on the derivation of fatigue load models in the UK National Annex.

Impact loads

Where accidents are caused by vehicles below the bridge, impact loads should be in accordance with BS EN 1991-1-7[21], see NA to BS EN 1991-1-7:2006, **NA.2.28 and NA.2.29**. Where accidents are caused by vehicles on the bridge, impact loads are given in BS EN 1991-2:2003 **4.7.3**.

Forces due to collision with vehicle restraint systems

The magnitude of forces transferred to a structure during vehicle collision against a vehicle restraint system depend 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, and the stiffness of the structure. For design purposes simple rules are adopted.

In BS 5400-2:2006, **6.7** the forces are based on containment level and mass of the structure. In BS EN 1991-2:2003, **4.7.3.3** the forces are based on the stiffness of the connection.

In the NA to BS EN 1991-2, **Table NA.6** the transverse forces correspond to the values given in BS EN 1991-2:2003, **Table 4.9 (n)** while the longitudinal and vertical forces are based on BS 5400-2:2006, **6.7.2** with partial factors set to unity. The approach recommended in the National Annex should assist the designers in determining whether the stiffness of a connection is weak or strong. It is envisaged that information relating to other containment levels may be developed in due course.

Abutments and wingwalls adjacent to bridge

Historically the vertical loads are given as a surcharge of 10 kN/m^2 for HA loading and 20 kN/m^2 for 45 units of HB loading in accordance with BS 5400-2:2006, **5.8.2.1**. However these empirical values are now considered out of date and it is best to require the designer to ascertain the load effect of a model vehicle using soil mechanics principles.

The model vehicle given in the NA to BS EN 1991-2:2003, **Figure NA.6** corresponds to the 32 tonne rigid vehicle amongst the set of vehicles given in BD 21/01 [22], **Table D1**.
In accordance with BS EN 1991-2:2003, **4.9.1(1) NOTE 2**, the dispersal of the loads through the backfill or natural ground should refer to BS EN 1997-1 [23]. The relevant information is given in PD 6694-1 [24], **6.6.3**. Further background is provided is provided by Shave et al[25].

Pedestrian loading

A pedestrian load intensity of 5 kN/m^2 is given as a characteristic value in EN 1991-1-7:2006, **5.3.2.1** and has been adopted in the NA to BS EN 1991-2:2003, **NA.2.36**. Although the nominal load in BS 5400-2:2006, **7.1.1** is still 5 kN/m^2, the pedestrian loading has effectively been reduced in the NA to BS EN 1991-2:2003, since higher partial load factors are used in BS 5400-2:2006.

It should be noted that a pedestrian load intensity of 5 kN/m^2 is considered to represent a very dense crowd. Values greater than 7 kN/m^2 would be inconceivable. Therefore the reduction in pedestrian load intensity is considered reasonable.

Footbridge vibration and acceptance criteria

In the light of the pedestrian excited vibration of the London Millennium Bridge in June 2000, there were considerable interests in the subject. Unfortunately there is not enough detailed design guidance given in EN 1991-2:2003, **5.7**. The dynamic models for pedestrian loads on bridges and acceptance criteria are given in the NA to BS EN 1991-2:2003, **NA.2.44**. Further information is given in several papers presented in Footbridge 2005 and Footbridge 2008 conferences[26] [27] [28], and reports written by Flint & Neill[29] and TRL[30].

Acknowledgements

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PROVISIONS OF THE UK NATIONAL ANNEX FOR EN 1991-2 FOR RAIL BRIDGES

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Abstract

BS EN 1991-2 defines models of traffic loads for the design of bridges including railway bridges. Where National Choice is permitted in BS EN 1991-2, the national decisions are presented in the UK National Annex for BS EN 1991-2.

Where existing practice based on BS5400-2 is still relevant, this has been perpetuated, an example being the loading required for traction and braking. In some cases, loading requirements are not dealt with in detail in BS EN 1991-2 but the option is given to provide additional requirements in its National Annex. In many cases, the National Annex allows such issues to be dealt with on a project specific basis. Examples include the design of deck plates and local elements to resist derailment actions from rail traffic.

BS EN 1991-2 also introduces a number of areas of railway bridge design that were not previously covered in BS 5400-2. These include the introduction of a factor (α) to account for railway loads greater or less than load model LM71 (equivalent to RU), groups of loads that represent the vertical loads with secondary horizontal loads as a single action,
the introduction of a High Speed Load Model (HSLM) for railway vehicles travelling at speeds in excess of 200km/h and the requirement to check the combined response of the structure and track.

This paper describes the alterative load models defined in the UK National Annex to BS EN 1991-2 and the values and approaches to be used where new concepts have been defined in BS EN 1991-2.

Notation

α	Load classification factor
Φ	Dynamic factor
ψ	Combination factor (BS EN 1990)
L_Φ	Determinant length
HSLM	High Speed Load Model
LM71	Load model 71 (equivalent to BS 5400-2 RU load model)
γ_{det}	Additional 10% allowance on the effects from live loads (NR/L2/CIV/020)
TSI	Technical Specification for Interoperability
TEN	Trans European Network
CWR	Continuous welded rail

Introduction

BS EN 1991-2[1] defines models of traffic loads for the design of bridges including railway bridges. Where National Choice is permitted in BS EN 1991-2, the national decisions are presented in the UK National Appendix for BS EN 1991-2[2]. The UK National Annex was developed on the basis of work undertaken for Network Rail (NR) and the Railways Safety and Standards Boad (RSSB). It also takes into account the background information provided through International Union of Railways (UIC) leaflets and from UIC research undertaken by Organisation for Research and Test (ORE) and European Rail Research Institute (ERRI).

Where existing practice based on BS5400-2[3] is still relevant this has been perpetuated, an example being the loading required for traction and braking. In some cases, loading requirements are not dealt with in detail in BS EN 1991-2 but the option is given to provide additional requirements in its National Annex. In many cases, the National Annex allows such issues to be dealt with on a project specific basis (design of deck plates and similar local elements to resist derailment actions from rail traffic for example). Differences in the loading requirements for traction and braking have been highlighted in an RSSB study to compare the provisions of BS 5400-2 with BS EN 1991-2.

There are a number of areas of railway bridge design that were not previously covered in BS 5400-2 and these require some explanation:

1. The application of High Speed Load Models (HSLMs) where trains travel at speeds in excess of 200 km/h.

2. Provisions for dynamic analysis of railway bridges.

3. Requirements for the combined response of structure and track to variable actions.

In some cases, design practice is different and some explanation will help understanding the differences and the impact on design (load classification factor and design for fatigue for example).

Choices have been made in the UK National Annex to BS EN 1991-2 where permitted and where possible. Where a choice has not been possible, the choice is left to be decided on a project-specific basis (derivation of aerodynamic actions from passing trains for example). This is either because the particular circumstances of the project need to be taken into account or the information is not yet available.

In most cases, where the UK National Annex to BS EN 1991-2 makes a choice or defines a value to use, it also allows alternative requirements or values to be decided for an individual project. Where this option is invoked, details will be included in Network Rail's Design Standard NR/L2/CIV/020[4] (planned for publication Spring 2011) or agreed in the Approval in Principle document for each bridge project.

New Concepts in EN 1991-2

A number of new concepts and load models have been defined in BS EN 1991-2 that were not routinely considered in railway bridge design to BS 5400-2. In many instances the UK National Annex to BS EN 1991-2 gives alternative approaches or values where national or individual project choice is permitted.

Heavy load model, SW/2

A new load model for heavy rail traffic, SW/2, has been introduced to reflect the very heavy vehicles that use some rail networks in Europe (Scandinavia for example). Existing load models cover mixed traffic (including freight) and are sufficient for UK purposes. Therefore UK National Annex to BS EN 1991-2, **NA.2.49** states that there is generally no requirement to design for SW/2 in the UK although in exceptional circumstances, particular heavy load requirements may be specified on a project-specific basis.

Load classification factor, α

LM71 represents the static effect of standard rail traffic operating over the standard gauge European railway network. Provision is made for varying the specified loading to account for different types, volumes and weights of rail traffic represented by LM71 and SW/0 through the introduction of a factor, α. For international lines a value of α not less than 1,10 is recommended in BS EN 1991-2:2003, **6.3.2.(3)P** and this is the value stated in UK National Annex to BS EN 1991-2, **NA.2.48**.

UK National Annex to BS EN 1991-2, **NA.2.48** allows alternative factors for the individual project. To enable Network Rail to achieve the in-service performance requirements set out in the High Speed Technical Specification for Interoperability (HS TSI) for structures, Network Rail require an additional partial factor γ_{det} to be used in the design of all bridges on routes that form part of the trans-European high-speed rail network (TENs routes). γ_{det} will provide an additional 10% on the effects of live loads at the ultimate limit state to allow for future deterioration. Rather than introduce a new factor, the application of γ_{det} has been included in Network Rail's choice of α in NR/L2/CIV/020 where the required value of α is stated as 1,21.

When α is applied to the characteristic values of LM71, the resulting loads are called "classified vertical loads".

Dynamic effects (including resonance)

In certain circumstances (bridge type, train speed), the quasi-static analysis, where the static classified vertical load effects are factored to account for the dynamic effect of the loading, is not appropriate and a dynamic analysis may also be required to establish the amount of dynamic enhancement of static load effects. BS EN 1991-2, 6.4 describes the procedure to check that resonance does not occur (as this could lead to unacceptable deformations and a reduced fatigue life) and the deck response (acceleration) is within the limits set out in BS EN 1990, **Annex A2[1]** (see Figure 1).

K National Annex to BS EN 1991-2, **NA.2.50** provides the flow charts to determine whether a dynamic analysis is required: **Figure NA.12** is applicable for simple structures (i.e. those that exhibit only longitudinal line beam behaviour) and **Figure NA.13** is applicable for both simple and complex structures (i.e. those that require deck/floor elements to distribute axle/wheel loads to primary longitudinal elements). The main difference between the flow charts in the UK National Annex to BS EN 1991-2 and BS EN 1991-2, is that the relevant authority, (for example Network Rail), have to accept decisions made about whether a dynamic analysis is required. A dynamic analysis will always be required where the line speed is greater than 200km/h or if the structure is outside the natural frequency limits in **Figure NA.14** of the UK National Annex to BS EN 1991-2.

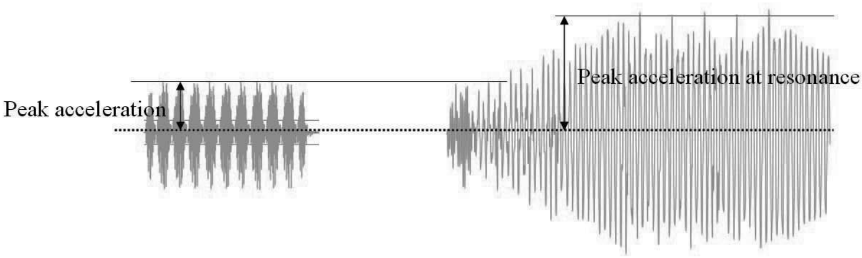


Figure 1. Peak acceleration performance

The UK National Annex to BS EN 1991-2, **NA.2.50** allows all choices and options relating to dynamic analysis to be specified for the individual project. Where a dynamic analysis is necessary, reference to NR/L2/CIV/020 is required. Much of the information contained therein was established following research undertaken for the West Coast Mainline Upgrade programme.

High speed load models (HSLM)

Where a dynamic analysis is required, the designer is required to check the effect of real trains (individual project to specify axle loads and spacings) and, where the route is part of the high speed Trans European Network (TENs route), also the High Speed Load Models (HSLMs). Two models, HSLM A and HSLM B, are defined in BS EN 1991-2, **6.4.6.1.1(4)** and **(5)** respectively (see Figures 2 and 3). Each HSLM represents a Universal Train with variable coach lengths. The pair of HSLMs together represent the dynamic load effects of articulated, conventional and regular, high speed passenger trains. These load models have been specified as requirements for design of structures on high speed TENs routes in the European High Speed Technical Specification for Interoperability for Infrastructure (HS INS TSI). UK National Annex to BS EN 1991-2, **NA.2.54** allows the individual project to specify additional requirements for the application of the HSLM but these are to be specified for the individual project. The background to the introduction of the HSLMs and design verification procedure is discussed in the paper "Design Performance Requirements for Rail Bridges in BS EN 1990:2002 Annex A2".

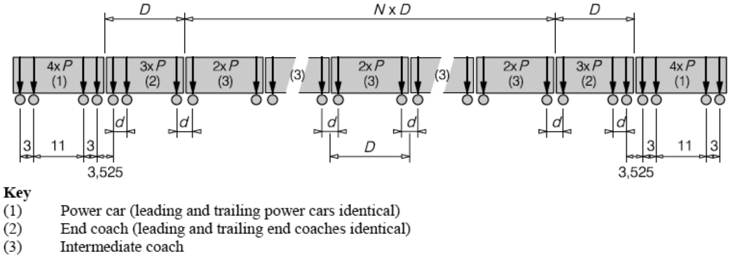


Figure 2. High Speed Load Model A (Figure 6.12)

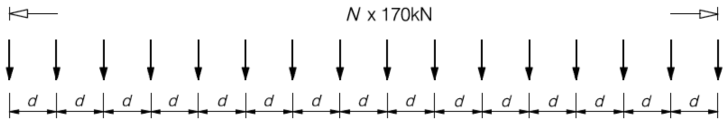


Figure 3. High Speed Load Model B (Figure 6.13)

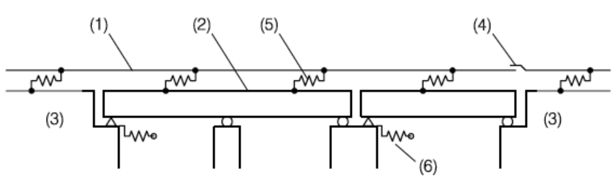
Combined response of structure and track

BS EN 1991-2, **6.5.4.1(3)P** requires the combined response of the structure and the track to railway loading and the resulting deformations to be taken into account to:

1. Ensure the rail stresses are not exceeded due to bridge deformations due to thermal variations, vertical loading, shrinkage and creep.
2. Ensure the rails and fixed bearings (where provided) can resist the longitudinal actions due to traction and braking.
3. Ensure that the track does not restrain the structure from behaving as required.

BS EN 1991-2, **6.5.4.5**, provides the design criteria in terms of allowable rail stresses and maximum structure deformations, and guidance on how to model the system and calculate the response of the combined structure and track response (see Figure 4).

Modelling of the combined track/structure system is permitted for complex structural arrangements and BS EN 1991-2, **6.5.4.4** provides guidance on calculation of the parameters required. Values of spring stiffnesses and initial displacements representing the track movement relative to the sleeper or structure are not specified with the option for the UK National Annex to BS EN 1991-2 or individual project to choose invoked.



Key
(1) Track
(2) Superstructure (a single deck comprising two spans and a single deck with one span shown)
(3) Embankment
(4) Rail expansion device (if present)
(5) Longitudinal non-linear springs reproducing the longitudinal load/ displacement behaviour of the track
(6) Longitudinal springs reproducing the longitudinal stiffness K of a fixed support to the deck taking into account the stiffness of the foundation, piers and bearings etc.

Figure 4. Modelling of track/structure system (Figure 6.19)

BS EN 1991-2, **6.5.4.6** provides helpful guidance on calculation methods for less complex structures. The guidance is based on UIC leaflet 774-3R[6].

UK National Annex to BS EN 1991-2, **NA.2.71** and **NA.2.72** allow the individual project to determine alternative calculation methods. Through these clauses Network Rail has identified when the combined response must be verified numerically. Where the deformation limits of BS EN 1990:2002 Annex A2 are met, NR/L2/CIV/020 states that in the following cases the combined response of the structure and the track may be deemed to be covered by the loading specified in BS EN 1991-2 and its National Annex:

1. Bridge with a total length up to 75 m, but subject to single span lengths not exceeding 50 m, and carrying ballasted or non-ballasted continuous welded rail (CWR) track with expansion switches provided where required by track construction standards,
2. Bridge comprising a single simply supported span up to 30 m expansion length, carrying ballasted CWR track without expansion switches,
3. Two-span simply-supported or continuous bridge with each span up to 30 m expansion length, carrying ballasted CWR track without expansion switches, provided that the fixed point for expansion is at the intermediate support,
4. Single-span bridge up to 15 m expansion length, carrying non-ballasted CWR track without expansion switches,
5. Two-span simply-supported or continuous bridge with each span up to 15 m expansion length, carrying non-ballasted CWR track without expansion switches, provided that the fixed point for expansion is at the intermediate support,
6. All bridges carrying jointed track; however, the rail joints shall be kept clear of the bridge as set out in track construction standards

Where assessment of the combined response of the structure and track is required, NR/L2/CIV/020 specifies the BS EN 1991-2 approach and provides values for the model parameters.

Groups of loads

In BS 5400-2 the designer was required to manually combine the primary railway live loads (vertical forces) with the applicable secondary live loads (traction, braking, centrifugal force and nosing force). This approach may continue for design using the Eurocodes, but BS EN 1991-2, **6.8.2** also introduces pre-defined groups of primary and secondary railway loads which are considered to act simultaneously. These load groups are summarised in BS EN 1991-2, **Table 6.11** (reproduced as Table 1 below).

number of tracks on structure			Groups of loads			Vertical forces			Horizontal forces			Comment
			Reference EN 1991-2			6.3.2/6.3.3	6.3.3	6.3.4	6.5.3	6.5.1	6.5.2	
1	2	≥ 3	number of tracks loaded	Load Group (8)	Loaded track	LM 71 (1) SW/0 (1), (2) HSLM (6)(7)	SW/2 (1),(3)	Unloaded train	Traction, Braking (1)	Centrifugal force (1)	Nosing force (1)	
			1	gr11	T_1	1			1 (5)	0,5 (5)	0,5 (5)	Max. vertical 1 with max. longitudinal
			1	gr 12	T_1	1			0,5 (5)	1 (5)	1 (5)	Max. vertical 2 with max. transverse
			1	gr 13	T_1	1 (4)			1	0,5 (5)	0,5 (5)	Max. longitudinal
			1	gr 14	T_1	1 (4)			0,5 (5)	1	1	Max. lateral
			1	gr 15	T_1			1		1 (5)	1 (5	Lateral stability with "unloaded train"
			1	gr 16	T_1		1		1 (5)	0,5 (5)	0,5 (5)	SW/2 with max. longitudinal
			1	gr 17	T_1		1		0,5 (5)	1 (5)	1 (5)	SW/2 with max. transverse
			2	gr 21	T_1 T_2	1 1			1 (5) 1 (5)	0,5 (5) 0,5 (5)	0,5 (5) 0,5 (5)	Max. vertical 1 with max longitudinal
			2	gr 22	T_1 T_2	1 1			0,5 (5) 0,5 (5)	1 (5) 1 (5)	1 (5) 1 (5)	Max. vertical 2 with max. transverse
			2	gr 23	T_1 T_2	1 (4) 1 (4)			1 1	0,5 (5) 0,5 (5)	0,5 (5) 0,5 (5)	Max. longitudinal
			2	gr 24	T_1 T_2	1 (4) 1 (4)			0,5 (5) 0,5 (5)	1 1	1 1	Max. lateral
			2	gr 26	T_1 T_2	 1	1		1 (5) 1 (5)	0,5 (5) 0,5 (5)	0,5 (5) 0,5 (5)	SW/2 with max. longitudinal
			2	gr 27	T_1 T_2	 1	1		0,5 (5) 0,5 (5)	1 (5) 1 (5)	1 (5) 1 (5)	SW/2 with max. transverse
			≥3	gr 31	T_i	0.75			0.75 (5)	0.75 (5)	0.75 (5)	Additional load case

Table 1. Groups of loads for rail traffic (Table 6.11 – Assessment of Groups of Loads for rail traffic (characteristic values f the multicomponent actions))

Each load group should be considered as a single action equivalent to the collective effects of the individual load components (i.e. a multi-component action), and applied in combination with appropriate non-traffic actions in accordance with BS EN1990, Annex A2. The benefits from using the groups of load include simpler automation when applying the railway loads in computer analyses and simplification when combining with other non-traffic actions. BS EN 1990, Annex A2, **A2.2.4(7)** infers that where the individual traffic load components are combined, the vertical and components of the railway load are considered separately. This is incorrect as throughout the Eurocodes, all actions are considered from a single source.

The factors used for the groups of loads (1,0, 0,5 or zero) were established in connection with work undertaken by a UIC Working Party formed to consider safety factors in advance of the publication of ENV 1991-3. They are probabilistic factors and not the same as the combination factors in BS EN 1990, **Table A2.3**. However, UK National Annex to BS EN 1991-2, **NA.2.79** advises that where economy is not adversely affected, the recommended factors may be increased to 1.0 to simplify the design process.

Alternative and Additional Load Models

A number of clauses in BS EN 1991-2 allow alternative load models with associated combination rules to be defined in the UK National Annex to BS EN 1991-2. Load models from the previous British Standards (BS 5400-2 for example) have been utilised where they remain relevant.

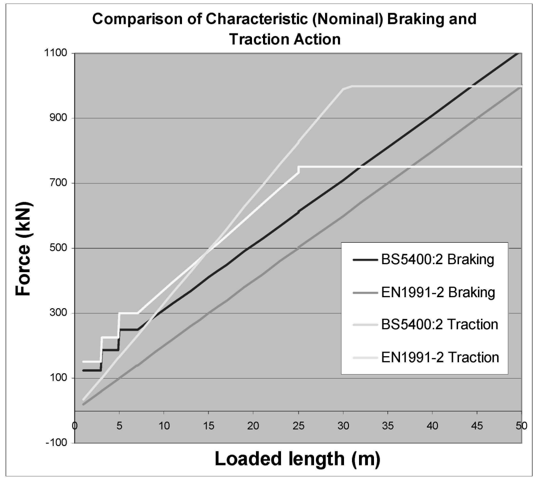


Figure 5. Traction and Braking in BS 5400-2 and BS EN 1991-2

Traction and braking

RSSB studies T696[7] and T741[8] examined the differences between the values in BS 5400-2 and BS EN 1991-2, **6.5.3** (see Figure 5). The UK National Annex to BS EN 1991-2, has taken the opportunity to include the provisions for traction and braking in BS 5400-2 in **NA.2.45.2.**

Braking: The values for braking in the NA to BS EN 1991-2, **Table NA.12** are greater than the BS EN 1991-2 values. BS EN 1991-2 sets a limit for the maximum value of the characteristic braking force of 6000 kN (this equates to a maximum loaded length of 300m). No such cut off exists in BS 5400-2 and the braking forces are higher throughout the loaded length range.

Traction: The values for traction in UK National Annex to BS EN 1991-2, **Table NA.12** are greater than the BS EN 1991-2 values for spans less than 14.7m. Above 14.7m the BS EN 1991-2, **6.5.3** values are greater. The maximum characteristic traction force, based on the UK National Annex to BS EN 1991-2, is 750kN compared to 1000kN for calculation using BS EN 1991-2,.**6.5.3.**.

RSSB study, T741 recommended that the BS 5400-2 traction and braking load models are specified in the UK National Annex to BS EN 1991-2, **NA.2.45.2** as alternative load models as permitted by BS EN 1991-2, **6.1(2)**. This is because, during drafting for BS EN 1991-2, it was recognised that provision should be made for load models that are outside the scope of those specified. In this case, the assumptions for vehicle traction and braking characteristics appear to be different for BS 5400-2 and BS EN 1991-2 and this was highlighted in an RSSB report which summarises the background to railway traffic actions. In the interim, before further studies can confirm the best approach to determining relevant traction and braking characteristics for current railway traffic, UK National Annex to BS EN 1991-2, **NA.2.45.2** requires that the greater of equations 6.20 and 6.21 in BS EN 1991-2 or, the values for traction and braking in the UK National Annex to BS EN 1991-2, are used.

Derailment

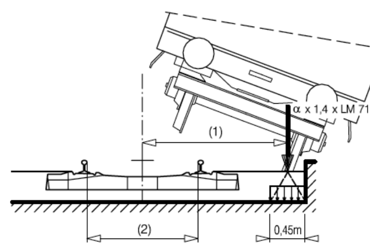
BS EN 1991-2, **6.7** requires the resulting actions from derailed trains on bridge decks to be considered. The scope is limited to actions on the bridge deck only and is intended to restrict damage to a minimum. UK National Annex to BS EN 1991-2, **NA.2.75** and **NA.2.76** specify additional checks required for the design of UK rail bridges.

Re-railing derailed trains

The UK National Annex to BS EN 1991-2, **NA.2.75** requires all deck plates and similar local elements to be designed to resist a series of vertical point loads that typically arise from jacking equipment to enable derailed trains be re-railed. These elements should be designed to support a concentrated load of $\alpha \times 1.4 \times 250$ kN (where α has a minimum value of 1,0) applied anywhere on the deck plate or local element. No dynamic factor is required to be applied to this design load. BS EN 1991-2, **6.7.1.(2)P** allows alternative loads and Network Rail Standard NR/L2/CIV/020 requires these loads to be applied on a 150 mm x 150 mm area anywhere on the deck between the robust kerbs.

Derailment situation II

BS EN 1991-2, **6.7.1.(2)P** allows alternative loads for the derailment situation to be specified or alternatively for the individual project (see Figure 6). NR/L2/CIV/020 requires that when the designer is checking the structure against overturning and instability at the ultimate limit state, the requirements of BS EN 1991-2, **6.7.1** shall be complied with, except that the 250 kN point loads of LM71 are not included.



Key
(1) Load acting on edge of structure
(2) Track gauge *s*

Figure 6. Design Situation II (Figure 6.27)

Robust kerbs

Prior to the withdrawal of GC/RC5510[9], it was recommended that Underline Bridges be provided with robust kerbs to contain the wheels of derailed vehicles, or alternatively, girders which are designed to perform this function, as a means of complying with the requirements of Office of Railway Regulation: Safety Principles and Guidance Part 2: Section A: Guidance on the Infrastructure, Chapter 4. The UK National Annex to BS EN 1991-2, **NA.2.76** allows additional measures to mitigate the consequences of a derailed train on the bridge, to be specified for the individual project. The 154kN design load (i.e. γ_{f3} x γ_{fL} x 100kN) previously defined for the design of robust kerbs in GC/RC5510 has been introduced in NR/L2/CIV/020.

Walkway load model

BS EN 1991-2, **6.3.7** provides a load model for non-public footpaths. The UK National Annex to EN 1991-2, has utilised the opportunity provided in BS EN 1991-2, **6.1(2)** to specify additional requirements for non-public footpath actions. UK National Annex to BS EN 1991-2, **NA.2.45.1** utilises the recommendations from GC/RC5510 to provide an additional 1kN/m load to be applied where cables are provided on the bridge and specifies the area over which the 2,0kN point load defined in EN 1991-2, **6.3.7** should be applied. UK National Annex to BS EN 1991-2, **NA.2.45.1** also provides an additional requirement to check for the application of a 1,0kN point load where it is more onerous than the 2,0kN load on a 100mm diameter circular area, which is more appropriate for the design of local elements. In addition to the vertical loads, UK National Annex to BS EN 1991-2, **NA.2.45.1** also specifies the handrail loads that were previously defined in GC/RC5510.

Aerodynamic actions

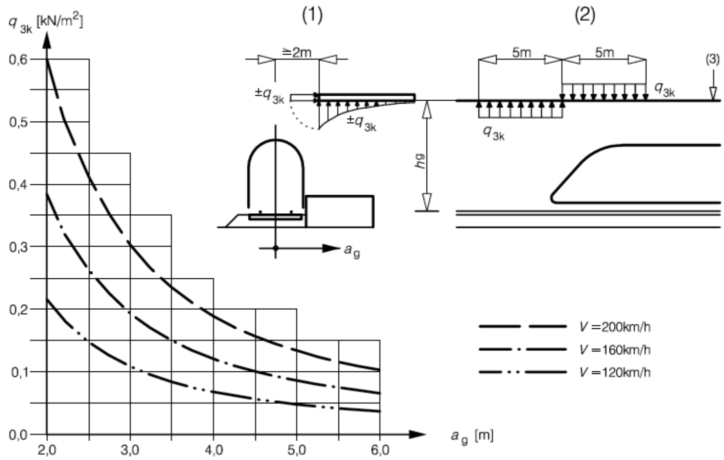
BS EN 1991-2, **6.6** requires the aerodynamic actions from passing rail traffic to be considered where this is likely to have a significant effect on structures adjacent to and over the track, such as:

1. A footbridge,
2. A station canopy or similar structure,
3. Parapets of an underline bridge,
4. Cladding panels attached to the bridge or other structure,
5. Noise or wind barriers attached to underline bridges, other structures or located adjacent to the track.
6. Platform structures.
7. Temporary structures adjacent to the track.

Structures that are susceptible to the aerodynamic effects of passing trains shall be designed to resist the resultant aerodynamic forces (see Figure 7).

Figure 7. Aerodynamic pressures on overbridges

BS EN 1991-2 provides graphs of speed related pressure changes due to passing trains for a range of vehicle types and structures (see Figure 8).



Key
(1) Section
(2) Elevation
(3) Underside of the structure

Figure 8. Aerodynamic pressure changes with distance from track (Figure 6.24 – Characteristic values of actions q_{3k} for simple horizontal surfaces adjacent to the track)

The UK National Annex to EN 1991-2, **NA.2.74** permits the loading to be determined for the individual project. This is because RSSB research project T750[10] has identified that these pressure changes are conservative for UK conditions. Some research was previously undertaken for Railtrack by the former BR Research and this forms the basis of the requirements in the UK National Application Document for ENV1991-3[11]. The requirements of this document are also included in NR/L2/CIV/020. These requirements are an improvement on the provisions in BS EN 1991-2 for UK infrastructure but they have limited scope and application. Further research by RSSB is underway to develop new design pressure curves to enhance or replace the ones in BS EN 1991-2. It is intended that these will be included in future updates of the UK National Annex to BS EN 1991-2.

Differences between EN1991-2 and BS5400-2

The requirements of BS EN1991-2 are not particularly different to BS 5400-2, as might be expected, as railway bridge design requirements have been consistent for many years, thanks to the high degree of European collaboration achieved through the UIC. The UK National Annex has been developed to maintain existing safety margins and established good practice wherever possible. However, there are some areas where the load models and the factors used to calculate the characteristic load effects, are not the same.

Value of dynamic factor, Φ

The design values of the effects from applying the RU or LM71 (static) load model representing typical trains that use the network, must be factored to account for the dynamic effect of the loading. This quasi-static concept was introduced by the UIC in 776-1R[12] and, subject to the fundamental frequency of the bridge being within the prescribed limits in UIC 776-3R[13], has been the design approach for many years. BS 5400-2 required the designer to check that deflection limits were in accordance with UIC 776-3 instead of checking the natural frequency. BS 5400-2 required the application of the dynamic factor Φ_3 when designing elements subject to bending and the dynamic factor Φ_2 when designing elements subject to shear.

Although the formulae defining the dynamic factors are the same in BS EN 1991-2, **6.4.5.2(2)** as in BS 5400-2, the definitions differ: BS EN 1991-2 does not differentiate between shear and bending, but distinguishes between the quality of track maintenance, i.e. carefully maintained track (Φ_2) and standard maintenance (Φ_3). The UK National Annex to BS EN 1991-2, **NA.2.51** specifies that Φ_3 is used unless an alternative value is specified for an individual project, where it is known that track maintenance standards will be high.

However, BS EN 1991-2, **D2(2)** requires the use of Φ_2 for fatigue assessment. This is because, although the factored load is sometimes exceeded at SLS, the collective European Railway (UIC) experience, has found Φ_2 to be adequate for the determination of fatigue resistance.

Value of determinant lengths, L_Φ

The determinant length is required to calculate the dynamic factor. BS 5400-2, **Table 17** focuses primarily on the main girder and floor (cross girder) elements. The determinant length for other details (if not included in table 17) should be taken as the length of the influence line for deflection of the element in question.

BS EN 1991-2, **Table 6.2** gives many more examples covering a variety of deck and construction types, with different values specified for steel and concrete structures. The UK National Annex to BS EN 1991-2, **NA.2.52** gives modifications to that table:

1. Case 1.1 – Deck plate (for both directions) – the lesser of three times cross girder spacing or cross girder spacing + 3 m.
2. Case 2.1 – Deck plate (for both directions) – cross girder spacing + 3 m.
3. Case 5.7 – Longitudinal cantilevers. No additional information has been provided.

Network Rail undertook studies to determine these values and the UK National Annex to BS EN 1991-2, replicates the majority of the values found in BS 5400-2. However, some differences remain, namely end cross girders (trimmers) have L_Φ = 3,6m in BS EN 1991-2 compared to 4,0m in BS 5400-2. As for BS 5400-2, BS EN 1991-2, **6.4.5.3(2)** states that where no value in the table is present, the length of the influence line for deflection of the element being considered should be used. There is also an option to provide alternative values if it is considered to be appropriate.

Traffic loads for fatigue

As BS 5400-2, a fatigue assessment of the structure must be undertaken to ensure that the required design life specified in the UK National Annex to EN 1991-2, **NA .2.82**, of 120 years, will be achieved. The simplified procedure in BS 5400-10[14], **9.2** is similar to the (safe life) damage equivalence method in BS EN 1993-2[15], **9.5** where factors are applied to either the load effects obtained from applying RU/LM71 or, the allowable stress range for a particular detail. Where the design fatigue stress is less than the allowed value, the design detail is assessed to be able to achieve the required design life.

This approach assumes that the load effects of applying RU/LM71 load models, represent the service trains and traffic mixes included in the traffic spectra provided for fatigue assessment. The train types and mixes are not the same in BS5400-10 and BS EN1991-2. However, it is considered that the BS EN1991-2 spectra represent current and anticipated future UK rail traffic.

The requirements for checking fatigue require some explanation with regard to the load classification factor α. It might be construed from BS EN 1991-2, **6.9(3)** that the factor α should be applied to Load Model 71 for the purpose of checking fatigue because alternative traffic mixes are necessary to represent the real traffic (i.e. where a value of α greater than 1,0 is necessary). BS EN 1991-2, Annex **D.2(2)** is quite clear on this point because it states that stress ranges due to LM71 (and where required SW/0) should be calculated excluding α. The reference in BS EN 1991-2, **6.9(3)** to α is intended to apply to situations where LM71 (and where required SW/0), does not represent the traffic using the bridge to be designed and alternative traffic mixes need to be considered for design (i.e. not for checking fatigue resistance).

It is also worth noting that there are differences in the detail classes / categories, most notably where fatigue failure across the throat of a weld is considered. In BS 5400-10, this detail is class W and the allowable stress for $2x10^6$ cycles is 43MPa whereas the equivalent BS EN 1993-1-9[16] detail category is 36. This will lead to larger weld details.

There are also significant differences in the S-N curves: The current BS 5400-10 curve is bi-linear with no cut off limits (except where all stresses are below the non-propagating level) whereas the BS EN 1993-1-9 curves are tri-linear with cut off limits. This leads to significant differences in the calculated number of cycles to failure or a particular damage state.

Future Amendments to the UK National Annex to BS EN 1991-2

BS EN 1991-2, **6.1(3)** states that for particular types of railway, including light rail and tramway systems, no values for load models are provided. This is an omission noted by CEN and it is possible that other load models will be added to the EN in future. The UK National Annex to BS EN 1991-2, **NA.2.46** recommends that load models be established for the individual project (light rail for example). Currently, there is no provision in the UK National Annex to BS EN 1991-2 for design of bridges on light rail networks. The RL load model used for design of London Underground bridges is an appropriate model that has been used not only for London Underground bridges but also for design of bridges on other light rail networks. It is intended that it be added to the UK National Annex to BS EN 1991-2 at the first opportunity for revision.

The load models for normal rail traffic in BS EN 1991-2 are now around 30 years old. They were developed on the basis of the rolling stock that used the network at the time and do not necessarily reflect some of the faster/heavier passenger vehicles that are now permitted to use the European Rail Network. Consequently the European Railway Agency (ERA) has submitted a proposal to CEN for review of the existing load models in BS EN 1991-2 to ensure their continuing relevance for the design of railway bridges.

As mentioned in the section dealing with aerodynamic actions the existing aerodynamic pressure curves in BS EN 1991-2, **6.6** are conservative for UK infrastructure. The UK National Annex to BS EN 1991-2 will be revised to take account of research currently being undertaken by RSSB. Potentially, BS EN 1991-2 could be amended in the light of the UK findings.

Conclusions

The provisions in the UK National Annex to BS EN 1991-2 will be familiar to the railway bridge designer who will not notice significant differences compared to the existing requirements of BS5400-2. However, BS EN 1991-2 does introduce a number of areas of railway bridge design that were not previously covered in BS 5400-2. These include the introduction of a factor (α) to account for railway loads greater or less than load model LM71 (equivalent to RU), groups of loads that represent the vertical loads with secondary horizontal loads as a single action, the introduction of a High Speed Load Model (HSLM) for railway vehicles travelling at speeds in excess of 200km/h and the requirement to check the combined response of the structure and track.

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OVERVIEW OF EARTHQUAKE DESIGN AND DEVELOPMENT OF UK NA FOR EN 1998-2 AND PD 6698

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Abstract

Damaging earthquakes are rare in the UK, though there are well recorded instances of them occurring. This is recognised in the UK National Forewords to the various parts of EN 1998, which state:

> 'There are generally no requirements in the UK to consider seismic loading, and the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998 need not apply. However, certain types of structure, by reason of their function, location or form, may warrant an explicit consideration of seismic actions.'

The introduction of EN1998-2 as a British Standard provided the necessity, and opportunity, to set out more formally advice on the situations where seismic design should be considered for bridges, and where needed, what the design procedures should be. The paper summarises the requirements of BS EN1998-2 for seismic design of bridges in areas of low seismicity and the supplementary guidance given in the UK National Annex to BS EN1998-2 and the BSI Published Document PD 6698:2009. The basis of the guidance for UK bridges is explained and the current statutory position is also described.

The possible implications for the design of major UK road and rail bridges are discussed; it is recognised that the recommendations will need to be reviewed in the light of experience after a suitable period of practical implementation.

Notation

G_k	characteristic value of a permanent action;
P_k	characteristic value of prestressing after all losses;
A_{Ed}	design seismic action;
Q_{1k}	characteristic value of the traffic load;
ψ_{21}	combination factor (quasi-permanent value) for traffic loads
Q_2	quasi-permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents etc.)

Introduction

Bridges where loss of serviceability would have a major regional or national economic impact are an example of the type of structure which, *prima facie*, might be thought to warrant seismic design, and there are a number of precedents for doing so in the UK, dating back many years (Cullen Wallace and Nissen[1], Mizon and Kitchener[2]). The issue of seismic design for structures in the UK raises difficult issues; although the low level of seismic activity in the UK has caused well documented cases of significant damage to buildings over many centuries and even a few deaths (Arup[3]), to the authors' knowledge there have been no recorded cases in the past hundred years of significant damage to well-built engineered structures of steel or concrete. This in itself justifies the common sense view that in the absence of special considerations, seismic actions need not be considered in the design of components of the built environment in the UK.

However, there is a consensus among seismologists that the UK, despite its low seismicity, may on rare occasions experience an earthquake of a magnitude (say M≥5) which is locally capable of producing potentially damaging motions. The probability of this occurring at any particular point in the UK is very low; in the language of probability theory, the hazard lies in the tail of the distribution (Figure 1). For facilities such as nuclear power plants or liquid natural gas (LNG) storage tanks, where the consequences of failure could be very adverse, there is general acceptance that the inclusion of seismic actions in the statutory requirements for design is, in principle, reasonable.

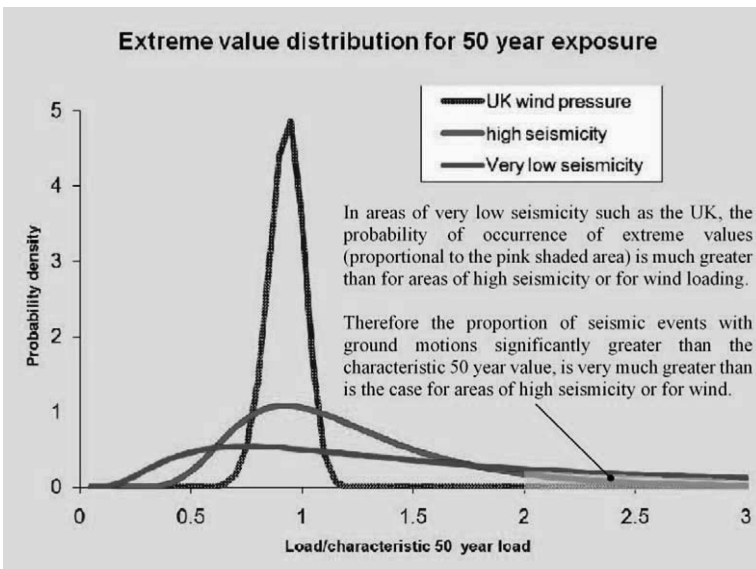

Figure 1: Extreme value statics for earthquake and wind loading

The issue becomes much harder to decide for other key elements of the UK infrastructure, such as bridges, forming vital communication links. What measures are reasonable to provide protection against rare earthquakes? Might the general provisions for robustness on their own provide a sufficient level of seismic protection? – that is, a level of seismic resistance which protects society with a level of reliability comparable to that provided against 'accidental' conditions such as terrorist action? Could simple design measures be devised which might reduce seismic vulnerability without significantly increasing design and construction costs, while increasing more generally the robustness and resilience of the structure? Might the variation in seismic hazard across the UK (well established in principle) and the differences in inherent seismic resistance between different structural forms allow a classification scheme which pinpointed particular cases where seismic design was warranted while exempting others?

During the period leading up to the introduction of the various parts of BS EN1998 as British Standards, a literature search and a wide consultation exercise, funded by the Institution of Civil Engineers and others, considered these issues and produced recommendations (Booth and Skipp[4]). A parallel exercise by the British Geological Survey in Edinburgh, reviewed the latest data on the level and spatial variation of seismic hazard across the United Kingdom (Musson & Sargeant[5]); this is described more fully in a companion paper (Lane *et al*[6]).

These two exercises formed the key inputs to the first drafts of the UK NA's to the various parts of BS EN 1998 and of the BSI Published Document PD6698. The rest of this paper outlines the advice provided on whether or not seismic actions need consideration for the design of UK bridges and, for situations where they are, summarises the consequences for design and detailing. Finally, the wider implications for the design of major UK bridges is discussed.

Situations Where Seismic Design Is Warranted for UK Bridges

As quoted above, the National Foreword to BS EN 1998-2 states that 'certain types of structure, by reason of their *function, location* or *form*, may warrant an explicit consideration of seismic actions' (italics added). PD 6698 discusses these three factors (namely function, location and form) as they apply generally to important structures in the following terms.

Influence of function

In some cases the function of a structure is such that failure due to very low probability events, including earthquakes, might need to be considered. At least four such categories of structure can be distinguished, as follows.

> 1) Structures where failure poses a large threat of death or injury to the population. Examples include nuclear power plants and major dams (both of which are explicitly outside the scope of BS EN 1998) and certain petrochemical installations, such as liquid natural gas (LNG) storage tanks and high pressure gas pipelines (which are within the scope of BS EN 1998).
>
> 2) Structures which form part of the national infrastructure and the loss of which would have large economic consequences. An example is a major bridge forming a transportation link vital to the national economy.
>
> 3) Structures whose failure would impede the regional and national ability to deal with a disaster caused by a major damaging earthquake.
>
> 4) Strengthening or upgrading of historic structures forming an important part of the national heritage.

In many cases, structures may fall into more than one category; for example, the seismic failure of a busy estuarial bridge might cause extensive human casualties, affect the regional or national economy and also impede the flow of disaster relief into the area affected by the earthquake.

PD 6698 advises that there is no need to consider seismic actions for the design of bridges in Consequence Classes CC1 and CC2 (according to BS EN1990). For bridges in Consequence

Class CC3, the need to design bridges for seismic actions should be considered on a project-specific basis. Factors to be considered include the safety, economic, social and environmental consequences of failure. Examples of bridges where the consequences of failure might be high enough for a seismic design to be considered are shown in Table 1. Such bridges do not necessarily require explicit seismic design, but should nevertheless be assessed to see if that need applies.

Table 1 – Examples of bridges with high consequence of failure where seismic design might need to be considered (from PD6698)

Factor influencing decision	Typical example
Economic impact	Bridges where loss of serviceability would have a major regional or national economic impact
Impact on post-earthquake relief	Bridges where loss of serviceability could have a major impact on the rescue effort or on aid delivery
Historic or cultural importance	Strengthening or upgrading of bridges which are an important part of the national heritage
Structural form (see Note)	Bridges that carry more than one level of traffic Bridges with suspension systems supporting spans over 50 m (see Note)

NOTE Certain types of bridge, including suspension bridges and historic bridges, are not included in the scope of BS EN 1998-2, so other sources of standards would be needed for their design.

The relationship between Consequence Class, Importance Class and Structure Category, is shown in Table 2, based on Interim Advice Note IAN 124/10[7].

Table 2. - Importance Classes of Highways Structures (from IAN 124/10[7])

				Comments
Structure Category in accordance with BD 2	0 & 1	2	3	Structure Categories are assumed to correspond to the Consequence Class as shown
Consequence Class EN1990 Table B1	CC1	CC2	CC3	For a whole structure
Importance Class EN1998-2 clause 2.1(4)P Note	IC I	IC II	IC II or IC III as agreed by TAA	Seismic design need not be considered for IC I and II. Technical Approval Authority is defined in BD 2.

Influence of location

The location of a structure affects the regional seismic hazard, which varies significantly across the UK, (Musson and Sargeant[3]); that is, the earthquake ground motions for a given annual probability of exceedence are significantly greater in some parts of the UK than others, although everywhere the hazard is very low by international standards. As discussed in the companion paper (Lane *et al* [3]), this may be allowed for by use of the seismic hazard map provided in Figure 2 of PD6698. Alternatively, site specific seismic hazard assessment may be carried out, in which case a return period for the ground motions may be chosen which is commensurate with the consequences of failure of the bridge in question, instead of the default value of 2,500 years which applies to the PD6698 map. Also, the site-specific assessment would account for the influence of local faults on the seismic hazard, which the PD6698 map would not. A site specific assessment may be the most appropriate choice for a major UK bridge in an area of higher than average seismicity.

Location also affects the local influences on seismic hazard and in particular, the effect of superficial soil deposits in modifying the seismic ground motions. As discussed in the companion paper (Lane *et al* [3]) the seismic response spectra provided in BS EN 1998-1 depend on the profile of the foundation soils involved, and thereby introduce an allowance for this effect.

Influence of structural form

All structures possess some degree of earthquake resistance, and this is greatly enhanced by the regulatory requirements to provide measures enhancing robustness, such as peripheral ties in buildings, detailing to increase ductility, and by the provision of wind and impact resistance. In many cases, these measures are considered to provide sufficient protection against seismic actions in the UK. In the context of bridge design, additional shear links, staggered splices, good tying in of steel, adequate bearing shelves, and similar measures can significantly improve structural performance in earthquakes for little additional cost.

By contrast, certain features can result in designs that are satisfactory for resisting wind or impact, but are vulnerable to seismic loading. Examples of such seismically unsatisfactory features in building structures are open and relatively weak ground storeys ('soft storeys'), very heavy roof masses and, large eccentricities between centres of mass and stiffness. Examples for bridges are bridge decks on bearings which provide poor lateral restraint and concrete bridge piers which are poorly confined by transverse reinforcement.

Decision on the need for seismic design of bridges

In the case of buildings, Booth and Skipp[3] propose a screening process for deciding whether or not seismic design is warranted; the screening process is not included in PD6698, nor the UK NA to EN 1998-1, but PD6698 does refer to it. It involves assessing three aspects of the seismic vulnerability of Importance Category 3 buildings, namely the level of the regional seismic hazard in comparison to the UK average, the presence or otherwise of particularly unfavourable structural features such as soft storeys, and the presence or otherwise of unfavourable soils such as soft soils. Where at least two of these three features are present, a seismic design is recommended, but where only one applies (for example above average seismicity but with good structural features and foundations soils) then it is suggested that an explicit seismic design is not warranted.

To the authors' knowledge, no such screening process has been proposed for bridges in the UK or indeed any other area of low seismicity. However, a similar process may be found helpful for UK bridges, weighing the influence of the bridge's *location* (particularly regional seismicity and local soils), its structural *form* and finally the *function* of the bridge in terms of the regional and national consequences of failure and the cost and time of repair. As experience develops in the application of EN 1998-2 to UK bridges, it would be valuable to develop further guidance on deciding on the need for seismic design, perhaps in collaboration with bridge engineers in other low seismicity areas of northern Europe. The current regulatory position in the UK is outlined in the section "Design requirements for seismic action in the UK" below.

Load Combinations for the Seismic Loadcase

The design value E_d of the effects of actions in the seismic design situation in EN 1998-2 is given by equation 1.

$$E_d = G_k "+" P_k "+" A_{Ed} "+" \psi_{21} Q_{1k} "+" Q_2 \quad (1)$$

where:

"+" implies "to be combined with";
G_k are the permanent actions with their characteristic values;
P_k is the characteristic value of prestressing after all losses;
A_{Ed} is the design seismic action;
Q_{1k} is the characteristic value of the traffic load;
ψ_{21} is the combination factor (quasi-permanent value) for traffic loads
Q_2 is the quasi-permanent value of actions of long duration (e.g. earth pressure, buoyancy, currents etc.) Actions of long duration are considered to be concurrent with the design seismic action.

Seismic action effects need not be combined with action effects due to imposed deformations (caused by temperature, shrinkage, settlements of supports, residual ground movements due to seismic faulting). An exception is the case of bridges in which the seismic action is resisted by elastomeric laminated bearings, where elastic behaviour of the system should be assumed and the action effects due to imposed deformations should be accounted for. Note that the displacement due to creep does not normally induce additional stresses to the system and can therefore be neglected. Creep also reduces the effective stresses induced in the structure by long-term imposed deformations (e.g. by shrinkage). Note also that wind and snow actions are not included with the seismic design situation.

Recommendations for Seismic Design and Detailing of UK Bridges

In cases where an explicit seismic design is considered necessary for Consequence Class CC3 bridges in the UK, the principal requirement is to carry out a seismic analysis and use its results to provide sufficient lateral resistance and deformation capacity. For bridges with a low fundamental lateral period of vibration, the lateral forces may be a substantial proportion of the structural mass (see Figure 2 in the companion paper by Lane *et al*, [3]), but for more flexible bridges relatively lower forces will apply. Simple equivalent static force analysis (fundamental mode analysis) may be sufficient where wind action effects comfortably exceed seismic ones, but more complex analysis methods (for example response spectrum, time history or non-linear static) are likely to be needed in other cases. The analysis requirements for cases not covered by BS EN 1998-2, in particular suspension bridges, would need particular attention, but generally the considerations for defining design motions and load combinations that apply to bridges within the scope of BS EN 1998-2 will apply.

For bridges in areas of moderate to high seismicity, providing sufficient lateral strength and deformation restraint capacity to decks is only one aspect of seismic design. An equally important aspect is to ensure adequate detailing. A crucial need is to identify the regions of the structure designed to yield during a severe earthquake, and to ensure that they are sufficiently ductile for the plastic deformation demands to which they may be subjected.

Bridges in areas of low seismicity are generally exempt from such considerations, because they are designed as limited ductility structures where significant yielding is not expected under actions due to the design earthquake. This greatly simplifies the design and detailing process, and is expected to be the adopted option for UK bridges. However, some simple measures can significantly increase ductility with relatively low impact on design effort and construction cost, and such measures provide a reserve of capacity in cases where seismic demands are greater than anticipated in design. BS EN 1998-2 requires a minimum set of such measures, which are endorsed by the UK NA to BS EN1998-2; they are outlined below. A possible way forward for the UK would be develop these minimum detailing rules to the extent where an explicit seismic analysis was necessary only in exceptional cases; if such simple rules were shown to have sufficiently low impact on cost and design effort, they might be extended to most or all Consequence Class 3 bridges in the UK, simplifying the decision making process for seismic design.

The minimum design and detailing rules in BS EN 1998-2, and the only ones additional to a seismic analysis that are required by the UK NA to BS EN1998-2, are as follows.

1. Shear strength of elements is provided assuming seismic actions corresponding to a behaviour factor of q=1 (instead of the more favourable value of 1.25 to 1.5 usually applying to the rest of the superstructure design) and the normal design shear resistance reduced by 1.25. This is to suppress shear failures from occurring before more ductile flexural failures. As discussed by Lane *et al*[3], the behaviour factor q is applied as a reduction factor to the calculated elastic seismic response, allowing for the reduction in response after the structure has yielded.
2. Foundations are designed for q=1, and the resistance calculated from the provisions of BS EN 1998-5. This is to suppress foundation failures in favour of yielding in the superstructure. Foundation failures may be difficult to detect and repair. They may also give rise to gross lack of alignment between piers and bridge deck, rendering the bridge unserviceable.
3. Non-ductile structural components, such as fixed bearings, sockets and anchorages for cables and stays and other non-ductile connections are also designed for q=1. This check may be omitted if it can be shown that the integrity of the structure is not affected by failure of such connections. The seismic design should also address the possibility of sequential failure, such as may occur in the stays of cable stayed bridges.
4. Minimum amounts of spiral or rectangular confining steel are required at potential plastic hinging points, defined as where the calculated bending demand is greater than the bending resistance divided by 1.3.

Other measures might also be considered to improve seismic performance. These include adding ‘lockup devices’ at bearings which allow thermal and other long term deformations but provide restraint to seismic movements. An option for suspension bridges is to release the end restraints at the towers, for example by providing either seismic ‘fuses’ which prevent excessive seismic loads being applied to the towers, or alternatively damped lateral buffer restraints to control seismic movements.

Design Requirements for Seismic Action in the UK

The Highways Agency intends to publish an Interim Advice Note IAN 124/10 which would state that the whole of the UK would be considered an area of very low seismicity. No formal advice has been published for the design of railway bridges to resist seismic actions.

Therefore the provisions of BS EN1998 need not apply for the design of bridges, unless otherwise specified by the Technical Approval Authority. Any site-specific seismic requirements (see PD6698), should be considered for the individual structure, where appropriate.

Implications for the Design of UK Bridges

In the great majority of cases, there will be no impact from the introduction of BS EN 1998-2 in the UK, since all Consequence Class CC1 and CC2 bridges and at least some (possibly most) Consequence Class CC3 bridges will not require any explicit seismic design.

Possible implications for bridges which do warrant seismic design might be as follows.

1. The significance of seismicity will tend to increase in relation to the ratio of the structural weight to the wind load. Generally, bridges with concrete and composite decks will be more greatly affected than those with steel decks.
2. Seismic loading may govern lateral strength requirements for foundations, particularly where piling through very soft materials.
3. Seismic loading may govern the design of restraint and displacement capacity at deck bearings.

Conclusions

The introduction of EN 1998-2 as a British Standard will not impact on the design of most UK bridges, since Consequence Class CC1 and CC2 bridges are not recommended as needing seismic design. PD6698 and IAN 124/10 provide some guidance on which Consequence Class CC3 bridges should be considered for an explicit seismic design. However, judgement will still be required, based on the severity of the consequences of failure of a particularly bridge, the local and regional level of seismicity and the inherent seismic resistance of the bridge's structural form. Developing further advice on these matters would be desirable and should be possible in the light of experience gained from the use of BS EN 1998-2.

For bridges that do warrant an explicit seismic design, a seismic analysis is required and some minimal level of seismic detailing. These are likely to have a particular impact on bridges where the ratio of structural weight to wind load is high, and in the presence of soft foundation soils. Seismic loading may also govern the design of restraint and displacement capacity at deck bearings.

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EN 1990 AND EN 1991 – PRACTICE PAPER: UNDERSTANDING COMBINATIONS OF ACTIONS

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Abstract
The purpose of structural analysis is to understand how a structure behaves under the various actions or loads which affect it. Understanding how these actions are combined is therefore fundamental to every analysis. There are significant differences in the way this is defined in the Eurocodes compared with BS5400, hence this may feel unfamiliar to many engineers. The objective of this paper is to illustrate how combinations of actions are applied in practice for bridges.

Notation
The notation used is in accordance with BS EN 1990: 2002, **1.6**. Key symbols are given below:

A_d	Design value of accidental action
E	Effect of actions
E_d	Design value of effect of actions
G_k	Characteristic value of permanent action
P	Relevant representative value of a prestressing action
Q	Variable action
$Q_{k,1}$	Characteristic value of leading variable action
$Q_{k,i}$	Characteristic value of the accompanying variable action i
γ_G	Partial factor for permanent actions
γ_P	Partial factor for prestressing actions
γ_Q	Partial factor for variable actions
ψ_0	Factor for combination value of a variable action
ψ_1	Factor for frequent value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action

Introduction
There are significant differences in the way actions for bridges are combined in the Eurocodes compared with BS 5400, hence this may feel unfamiliar to many engineers. In particular, the combinations of actions to be considered and the relevant load factors were explicitly defined in BD 37/01, **Table 1**[1]. In contrast, in the Eurocodes the designer is required to form and factor the actions based on a series of equations. The objective of this paper is to illustrate how combinations of actions are applied in practice for bridges.

These differences to past practice include: the need to select an appropriate set of partial factors depending on which ultimate limit state is being considered; the introduction of a new combination factor for variable actions; and, the use of six or more equations to combine actions. Further, these combination equations are less explicit in which actions need to be combined together, leading to the apparent need to consider many alternative combinations.

The features of these combination equations are presented, and the similarities and differences between them explored. Use of the different sets of partial factors is explained. Attention is drawn to some of the rules which reduce the number of combinations of actions which need to be considered, including the rules on traffic load groups.

A series of illustrations are developed which demonstrate how a designer combines actions in practice. These illustrations include: the use of different combination equations; the treatment of partial factors; the selection of appropriate leading and accompanying variable actions and corresponding factors; and, the use of load groups.

Combinations of Actions: Equations

The application of partial factors to produce load combinations is defined by a series of equations in BS EN 1990: 2002, **Section 6**. Highlighted below are three key equations for ULS and three for SLS.

Similar terms are present in each equation: permanent actions $G_{k,j}$, prestress action P; leading variable action $Q_{k,1}$; accompanying variable actions $Q_{k,i}$; and, accidental or seismic actions A_d or A_{Ed}. The differences between equations are the applied partial factors γ, choice of which representative value will be used through factor ψ and whether an accidental action term is included.

A distinction is made between leading and accompanying variable actions by BS EN 1990: 2002, **6.4.3.1(2)**. At most one variable action is termed the leading action, and the other variable actions which are considered to act simultaneously are reduced by a factor ψ. This takes account of reductions in design values of variable actions which occur simultaneously, i.e., that there is a reduced probability that various loadings acting together will all attain extreme values simultaneously.

The design value, E_d, is an effect caused by the action, such as moment, shear, stress, torsion, deflection, etc., and the 'effect of' term $E\{...\}$ usually represents the output from a structural analysis model.

ULS combination equations

The combinations of actions at ULS are defined in BS EN 1990: 2002, **6.4.3**, and are listed below. The choice of equation depends on the design situation being considered, in accordance with BS EN 1990: 2002, **3.2(2)**, i.e., persistent, transient, accidental or seismic. Of these, persistent relates to conditions of normal use (which includes, for example, loading by abnormal vehicles if this is a design requirement of the structure), whilst transient relates to temporary conditions applicable to the structure, such as during execution or repair.

ULS persistent / transient design situation, BS EN 1990: 2002, **6.4.3.2**:

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq1}\gamma_{\mathrm{G},j}G_{\mathrm{k},j} \text{ "+" } \gamma_{\mathrm{P}}P \text{ "+" } \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1}\gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i}\right\}$$

EN1990 (6.10)

For bridges, the alternative Equations 6.10a and 6.10b should not be considered (see the UK NA to BS EN 1990: 2002, **NA.2.3.7.1**).

ULS accidental design situation, BS EN 1990: 2002, **6.4.3.3**:

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq1}G_{\mathrm{k},j} \text{ "+" } P \text{ "+" } A_{\mathrm{d}} \text{ "+" } \left(\psi_{1,1} \text{ or } \psi_{2,1}\right)Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1}\psi_{2,i}Q_{\mathrm{k},i}\right\}$$

EN1990 (6.11b)

EN1990 suggests that the choice of which ψ factor (ψ_1 or ψ_2) to use for the leading variable action depends on the particular situation being considered, however, the UK NA to BS EN 1990: 2002, **Table NA.A2.5,** states that the frequent value should normally apply, i.e., the factor ψ_1 should be used and not ψ_2. Some exceptions to this are given in BS EN 1990: 2002, **A2.2.5**, for example, derailment of trains uses the combination value, i.e., ψ_0 is used.

ULS seismic design situation, BS EN 1990: 2002, **6.4.3.4**:

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq1}G_{\mathrm{k},j} \text{ "+" } P \text{ "+" } A_{\mathrm{Ed}} \text{ "+" } \sum_{i\geq1}\psi_{2,i}Q_{\mathrm{k},i}\right\}$$

EN1990 (6.12b)

This seismic combination equation is included for completeness. However, the UK National Foreword to BS EN 1998-1: 2004 explains that the whole of the UK may be considered an area of very low seismicity in which the provisions of EN 1998 need not apply other than for special types of structure.

SLS combination equations

The combinations of actions at SLS are defined in BS EN 1990: 2002, **6.5.3**, and are listed below. The choice of equation depends on the particular verification being undertaken and typically the combination to be used will be stated explicitly in the material parts EN1992 to EN1999.

SLS characteristic combination, BS EN 1990: 2002, **6.5.3(2)** a):

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq1}G_{\mathrm{k},j} \text{ "+" } P \text{ "+" } Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1}\psi_{0,i}Q_{\mathrm{k},i}\right\}$$

EN1990 (6.14b)

SLS frequent combination, BS EN 1990: 2002, **6.5.3(2)** b):

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq1}G_{\mathrm{k},j} \text{ "+" } P \text{ "+" } \psi_{1,1}Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1}\psi_{2,i}Q_{\mathrm{k},i}\right\}$$

EN1990 (6.15b)

SLS quasi-permanent combination, BS EN 1990: 2002, **6.5.3(2)** c):

$$E_d = E\left\{\sum_{j\geq1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i\geq1} \psi_{2,i} Q_{k,i}\right\} \qquad \text{EN1990 (6.16b)}$$

Summary of combination equations

It is instructive to compare the form of these equations and observe that the term in brackets can be rewritten in the following generalised form for each of the six equations given above:

$$\sum_{j\geq1} \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_P P \text{ "+" } [A_d] \text{ "+" } \gamma_{Q,1}\psi_{Q,1}Q_{k,1} \text{ "+" } \sum_{i>1} \gamma_{Q,i}\psi_{Q,i}Q_{k,i} \qquad (1)$$

The variation in the applied partial factors, γ and ψ, is summarised in Table 1 below:

	Equation	Permanent action $G_{k,j}$	Prestress P	Accidental or seismic A_d	Leading variable action $Q_{k,1}$		Accompanying variable action $Q_{k,i}$	
		$\gamma_{G,j}$	γ_P	A	$\gamma_{Q,1}$	$\psi_{Q,1}$	$\gamma_{Q,i}$	$\psi_{Q,i}$
ULS persistent/transient	6.10	γ_G	γ_P	0	γ_Q	1.0	γ_Q	ψ_0
ULS accidental	6.11b	1.0	1.0	A_d	1.0	ψ_1	1.0	ψ_2
ULS seismic	6.12b	1.0	1.0	A_{Ed}	1.0	ψ_2	1.0	ψ_2
SLS characteristic	6.14b	1.0	1.0	0	1.0	1.0	1.0	ψ_0
SLS frequent	6.15b	1.0	1.0	0	1.0	ψ_1	1.0	ψ_2
SLS quasi-permanent	6.16b	1.0	1.0	0	1.0	ψ_2	1.0	ψ_2

Table 1. Factors used for combinations of actions

The use of a value 1.0 where particular factors do not appear in certain equations corresponds with the note to BS EN 1990: 2002, **6.5.3(2)**, which explains that in the SLS equations the partial factors γ are equal to 1.

Hence the differences between these six combination equations concern:

- which values of partial factors γ_G, γ_P and γ_Q are applied;
- which representative values of actions are used;
- and, whether there is an accidental or seismic action involved.

Partial Factors for Actions (γ)

Partial factors for actions, γ_G, γ_P and γ_Q, are only applied at ULS and then only for the persistent/transient design situation. Comparing the SLS characteristic combination (row 4 of Table 1) with the ULS persistent/transient situation (row 1 of Table 1), it is apparent that the only difference between the two is the inclusion of the γ factors at ULS.

Values of the γ factors are obtained from Annex A to EN1990 as directed by BS EN 1990: 2002, **6.4.4(1)**. Annex A2 is used for bridges, and BS EN 1990: 2002, **A2.3.1(1)** states that values should be in accordance with Tables A2.4(A) to (C). It is crucial to observe that these are Nationally Determined Parameters, and that these complex tables are superseded in their entirety. The replacement tables are given by UK NA to BS EN 1990: 2002, **NA.2.3.7.1** which states that Tables NA.A2.4(A) to (C) should be used.

The three tables give different sets of γ factors depending on the ultimate limit state being considered, i.e., EQU, STR or GEO as defined in BS EN 1990: 2002, **6.4.1(1)**. It is therefore critical to understand which ULS is being verified as the choice leads to different partial factors.

The EQU limit state concerns loss of static equilibrium and requires the use of the Set A factors in Table NA.A2.4(A) as per BS EN 1990: 2002, **A2.3.1(3)**. Examples of such limit states are met infrequently in bridge design, examples being possible overturning during launched construction of a bridge deck, or during erection of a balanced cantilever.

Structural resistance and deformation are verified by the STR limit state. Where this does not involve geotechnical actions then Set B factors are used from Table NA.A2.4(B) as per BS EN 1990: 2002, **A2.3.1(4)**. This is a common design case, for example, design of deck members of a non-integral bridge.

If the strength of soil or rock is significant in providing resistance then the GEO limit state also applies. For GEO limit state and STR limit state verifications involving geotechnical actions both Set B and Set C factors from Tables NA.A2.4(B) and NA.A2.4(C) need to be applied in separate, parallel calculations and the design must satisfy the worse case. This is Design Approach 1 in BS EN 1990: 2002, **A2.3.1(5)**, as specified by the UK NA to BS EN 1990: 2002, **NA.2.3.7.2**. This is relevant for the design of foundations and also, for example, for the design of structural members such as footings, piles, wing walls and abutment walls.

The dual approach is used to account for uncertainty in a geotechnical variable (e.g., horizontal soil pressure) which can be dealt with either by applying the partial factor to the actions acting on the soil (e.g., uncertainty on soil density) or by applying the partial factor to the ground properties (e.g., uncertainty on angle of friction), or through a combination of the two.

Note that the Set C partial factors on actions are all lower than those in Set B. They are used in conjunction with an increased set of material partial factors, set M2, defined in the UK NA to BS EN 1997-1: 2004, **Table A.NA.4**. It will often be found that Set C governs the sizing of elements (e.g., footing size, wall thickness, etc.) while Set B governs the resistance of the element (e.g., determination of reinforcement within the section).

Combinations of actions for fatigue verifications at the FAT limit state are defined by further equations in the material-specific parts EN1992 to EN1999, as per the note to BS EN 1990: 2002, **6.4.1(1)**. An example is the damage equivalent stress method for concrete bridges given in Annex NN to BS EN 1992-2: 2002 and the load combination given in BS EN 1992-1-1: 2004, **6.8.3(3)**.

There is an important distinction in the use of a capitalised or lower case subscript on the partial factor (i.e., γ_f or γ_F), where the capitalised subscript indicates an implicit model factor has been included. BS EN 1990: 2002, **Figure C3** indicates two partial factors representing: uncertainty in representative values of actions, γ_f, and model uncertainty in actions and action effects, γ_{Sd}. These may be combined together as a single factor, γ_F, as per BS EN 1990: 2002, **6.3.2(2)**. The partial factors given in the UK NA to BS EN 1990: 2002 generally all take this combined form, i.e., no separate model factor is required. The partial factors given in BS EN 1990:2002 may sometimes appear high in relation to the statistical uncertainty in the representative value of an action alone (e.g., for steel self-weight), but this is because they also account for model uncertainty.

Representative Values of Variable Actions (ψ Factors)

Four representative values of a variable action are defined by BS EN 1990: 2002, **4.1.3**: characteristic, combination, frequent and quasi-permanent. The characteristic value Q_k of the action is defined in the relevant part of EN1991 and is a statistically extreme value. The other representative values are obtained by multiplying the characteristic value by a reduction factor, to obtain: the combination value $\psi_0 Q_k$; the frequent value $\psi_1 Q_k$; or, the quasi-permanent value $\psi_2 Q_k$. The combination, frequent and quasi-permanent values are less statistically extreme than the characteristic value, so ψ_0, ψ_1 and ψ_2 are always less than or equal to unity.

These ψ factors are used in all the combination equations. A ψ factor is always used in conjunction with the accompanying variable actions $Q_{k,i}$. A ψ factor is sometimes used with the leading variable action $Q_{k,1}$, depending on the combination being considered. The ψ factors only apply to variable actions Q_k and are never applied to permanent actions G_k.

Values of ψ_0, ψ_1 and ψ_2 are obtained for bridges from Annex A2 to BS EN 1990: 2002. Clause **A2.2.6(1)** states that recommended values are given in Tables A2.1 to A2.3. Again, it is crucial to observe that these are Nationally Determined Parameters. The UK NA to BS EN 1990: 2002, **NA.2.3.6.1** states that Table NA.A2.1 is used for road bridges and Table NA.A2.2 for footbridges. For rail bridges Table A2.3 from EN 1990 is used.

Selection of the appropriate value for ψ_0, ψ_1 or ψ_2 is thereafter straightforward, provided that the designer is clear which combination is being considered (e.g., SLS characteristic or SLS frequent). This depends on the particular verification being undertaken and will be stated explicitly by material part, for example BS EN 1992-1-1: 2004, **7.2(5)** requires the SLS characteristic combination, since it states (italics added), "Unacceptable cracking or deformation may be assumed to be avoided if, *under the characteristic combination of loads*, the tensile stress in the reinforcement does not exceed $k_3 f_{yk}$".

Illustrative Examples

Appendix 1 contains four illustrations which demonstrate how the above topics apply in practice, using the following aspects of an example bridge design:

Illustration 1: ULS bending check of main beam at midspan

- Selection of leading and accompanying variable actions
- Combination equation for the ULS persistent design situation
- Effects due to vertical loads from traffic load groups

Illustration 2: SLS stress check of main beam at midspan

- SLS characteristic combination

- Differences between ULS and SLS combination equations

Illustration 3: ULS check on pier

- Transverse loads from traffic load groups
- Combination value for traffic load groups (using ψ_0)
- Partial factors for favourable and unfavourable effects

Illustration 4: ULS check on pier for accidental impact

- Combination equation for accidental design situations
- Frequent value for traffic load groups (using ψ_1)

Conclusions

This paper has illustrated how combinations of actions are applied in practice. Key concepts which it is essential for designers to understand include:

- Design situations (persistent / transient / accidental);
- Differentiation between ultimate limit states (EQU / STR / GEO);
- Selection of the appropriate combination equation;
- Choice of leading and accompanying variable actions;
- Use of traffic load groups to reduce number of combinations needing to be considered;
- Treatment of traffic load groups at different representative values.

The application of these key concepts to combinations of actions has been illustrated by a number of examples. These illustrations included: the use of different combination equations; the treatment of partial factors; the selection of appropriate leading and accompanying variable actions and corresponding factors; and, the use of load groups.

References

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Appendix 1: Illustrative Examples

The way in which actions are combined for different limit states will be illustrated by four examples, all based on the following structure:

Consider a two-span steel-composite bridge carrying road traffic, continuous across the central pier and supported on bearings at the pier and abutments. There is transverse fixity at the central pier and abutments and longitudinal fixity at one abutment. A road passes below the bridge close to the pier.

The bridge is subject to the actions shown in Table 2. Note that in a real design situation other loads and/or effects may also need to be considered, e.g., differential settlement has not been considered; it would be treated as a permanent action in accordance with BS EN 1990: 2002, **A2.2.1(15)**. Likewise, shrinkage and creep should be considered in accordance with BS EN 1994-2: 2005, **5.4.2.2**.

Action		Symbol	Characteristic values		
			Midspan bending (kNm)	Reaction at pier (kN)	Transverse force (kN)
Dead load	(steel)	$G_{k,DLst}$	216	138	0
	(concrete)	$G_{k,DLconc}$	1218	777	0
Superimposed dead load		$G_{k,SDL}$	906	(max) 577 (min) 223	0
Accidental pier impact		A_d	-	-	1500 750
Wind		$Q_{k,W}$	51	24	236
Thermal		$Q_{k,Th}$	215	30	0
Road traffic (general)		$Q_{k,Tr}$	Values given for individual components:		
gr1a	Road traffic LM1	$Q_{k,TS}$ $Q_{k,UDL}$	2955 1378	413 356	0
	Pedestrian traffic	$Q_{k,Ped}$	82	52	
gr2	Road traffic LM1, coexisting with reaction	$Q_{k,TS}$ $Q_{k,UDL}$	226 1117	(max) 595 (max) 712	226
gr5	Road traffic LM3	$Q_{k,LM3}$	4154	1208	0
	Road traffic LM1, coexisting with LM3	$Q_{k,TS}$ $Q_{k,UDL}$	341 153	41 36	
gr6	Road traffic LM3, coexisting with reaction	$Q_{k,LM3}$	2098 3417	(max) 1642 (min) 1167	243
No prestress actions apply					

Table 2. Characteristic effects of actions on example bridge

The characteristic values are assumed to have been obtained by a linear elastic analysis, hence the design effects can be superposed outside of the analysis model, i.e.:

$$E_{\mathrm{d}} = E\{\gamma_{\mathrm{F},1}F_1 + \gamma_{\mathrm{F},2}F_2 + \ldots\} = \gamma_{\mathrm{F},1}E\{F_1\} + \gamma_{\mathrm{F},2}E\{F_2\} + \ldots$$

The superimposed dead load characteristic values incorporate the additional provisions for bridges given in EN 1991-1-1: 2002, **5.2.3**. These include an increase of 55% in the nominal thickness of the surfacing to allow for future recoating, as per the UK NA to EN 1991-1-1: 2002, **Table NA.1**. The minimum value given above corresponds to a 40% reduction in thickness.

Snow loading does not generally need to be considered for bridges in the UK, see UK NA to BS EN 1990: 2002, **NA.2.3.3.3**. The circumstances when snow loading may need to be considered are given in the UK NA to BS EN 1991-1-3: 2003, **NA.4.1.1**.

Road traffic loading consists of various components (e.g., horizontal and vertical, normal and special vehicles, etc.). These are combined into a single variable action, either leading or accompanying, in accordance with BS EN 1990: 2002, **A2.2.1(9)**. The mutually exclusive load groups, gr1a to gr6, which need to be considered, are defined in the UK NA to BS EN

1992-1:2003, **Table NA.3**, and the groups used to generate the global effects in the illustrations are summarised in Table 3 below:

Load group	Comment	LM1	LM3	Horizontal	Footway
gr1a	Max vertical, normal traffic	Characteristic	-	-	0.6 x Characteristic
gr2	Max horizontal, normal traffic	Frequent	-	Characteristic	-
gr5	Max vertical, special vehicles	Frequent	Characteristic	-	-
gr6	Max horizontal, special vehicles	-	Characteristic	Characteristic	-

Table 3. Summary of characteristic values of highway load groups used in illustrations

Illustration 1: ULS Bending Check of Main Beam at Midspan

This illustration considers:

- Selection of leading and accompanying variable actions
- Combination equation for the ULS persistent design situation
- Effects due to vertical loads from traffic load groups

Basis of design

This bending verification is a ULS persistent design situation, so Equation 6.10 is used. The partial factors γ need to be obtained; the set used depends on the limit state (EQU, STR or GEO).

The verification is of structural resistance and there are no geotechnical actions or loss-of-equilibrium considerations, hence the limit state is STR. This means that UK NA to BS EN 1990: 2002, **Table NA.A2.4(B)** is used, giving the Set B partial factors reproduced below in Table 4:

Action	Symbol	Partial factor	
		Unfavourable $\gamma_{G,sup}$ / γ_Q	Favourable $\gamma_{G,inf}$ / γ_Q
Dead load (steel)	$\gamma_{G,DLst}$	1.2	0.95
Dead load (concrete)	$\gamma_{G,DLconc}$	1.35	0.95
Superimposed dead load (SDL and surfacing)	$\gamma_{G,SDL}$	1.2	0.95
Traffic actions (LM1, LM3, pedestrian, etc.)	$\gamma_{Q,Tr}$	1.35	0
Wind	$\gamma_{Q,W}$	1.7	0
Thermal	$\gamma_{Q,Th}$	1.55	0

Table 4. Set B partial factors

Traffic as leading variable action

In this example the load effects due to traffic are the most significant variable action for midspan bending (Table 2 gives $Q_{k,LM3}$ = 4154 kNm for one component of traffic loading, compared with $Q_{k,Th}$ = 215 kNm for thermal and $Q_{k,W}$ = 51 kNm for wind). By inspection, setting traffic as the leading variable action will therefore generate the maximum design effects.

The other variable actions in this example are wind and thermal actions. These two actions do not need to be considered simultaneously, in accordance with UK NA to BS EN 1990: 2002, **NA.2.3.3.4**. Table 2 shows that thermal actions give a bigger effect on main beam bending than wind (215 kNm compared with 51 kNm), hence the thermal action is taken as the only accompanying action.

The worst midspan sag effect is generated by using the load groups which maximise vertical traffic loads, hence using load groups gr1a or gr5 (see UK NA to BS EN 1992-1: 2003, **Table NA.3** and Table 3 above). These two load groups involve Load Models LM1 and LM3 and pedestrian loading. For LM1, the worst sag is produced by positioning the tandem system close to midspan and applying the UDL to the adverse areas (i.e., one span loaded, other span unloaded) as per BS EN 1992-1:2003, **4.3.2(1) (b).** For LM3, the worst sag is produced by positioning the SV or SOV vehicle around midspan. In this example, it is not possible to tell immediately by inspection whether load group gr1a or gr5 will govern, hence both cases will be evaluated.

Combining actions using Equation 6.10:

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq 1}\gamma_{\mathrm{G},j}G_{\mathrm{k},j}\ \text{"+"}\ \gamma_{\mathrm{P}}P\ \text{"+"}\ \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1}\ \text{"+"}\ \sum_{i>1}\gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i}\right\} \qquad \text{EN1990 (6.10)}$$

Replacing the summations with the appropriate actions from Table 2, with traffic as the leading and thermal as the accompanying variable action:

$$E_{\mathrm{d}} = E\left\{\begin{matrix}\gamma_{\mathrm{G,DLst}}G_{\mathrm{k,DLst}} + \gamma_{\mathrm{G,DLconc}}G_{\mathrm{k,DLconc}} + \gamma_{\mathrm{G,SDL}}G_{\mathrm{k,SDL}} + \\ \gamma_{\mathrm{Q,Tr}}Q_{\mathrm{k,Tr}} + \gamma_{\mathrm{Q,Th}}\psi_{0,\mathrm{Th}}Q_{\mathrm{k,Th}}\end{matrix}\right\} \qquad (2)$$

The relevant γ partial factors are listed in Table 4 and can be substituted into Equation (2). In addition, the accompanying variable action requires a ψ_0 factor. For thermal actions, a value of ψ_0 = 0.6 is obtained from UK NA to BS EN 1990: 2002, **Table NA.A2.1**:

$$E_{\mathrm{d}} = E\left\{\begin{matrix}1.2G_{\mathrm{k,DLst}} + 1.35G_{\mathrm{k,DLconc}} + 1.2G_{\mathrm{k,SDL}} + \\ 1.35\left[Q_{\mathrm{k,Tr}}\right] + 1.55\cdot 0.6\cdot Q_{\mathrm{k,Th}}\end{matrix}\right\} \qquad (3)$$

Traffic load group gr1a will be considered first.

ULS midspan bending, traffic group gr1a

Load group gr1a consists of the characteristic value of LM1 and 0.6 times the characteristic value of pedestrian loading, from the UK NA to BS EN 1991-2: 2003, **Table NA.3**:

$$Q_{\mathrm{k,gr1a}} = Q_{\mathrm{k,TS}} + Q_{\mathrm{k,UDL}} + 0.6Q_{\mathrm{k,Ped}} \tag{4}$$

The entire load group is applied as the leading variable action so substituting $Q_{\mathrm{k,gr1a}}$ from Equation (4) for $Q_{\mathrm{k,Tr}}$ in Equation (3) gives:

$$E_{\mathrm{d,gr1a}} = E\left\{\begin{array}{l} 1.2G_{\mathrm{k,DLst}} + 1.35G_{\mathrm{k,DLconc}} + 1.2G_{\mathrm{k,SDL}} + \\ \quad 1.35\left[Q_{\mathrm{k,TS}} + Q_{\mathrm{k,UDL}} + 0.6Q_{\mathrm{k,Ped}}\right] + 1.55 \cdot 0.6 \cdot Q_{\mathrm{k,Th}} \end{array}\right\} \tag{5}$$

Inserting the characteristic values from Table 2 into Equation (5) gives the result shown in Table 5:

ULS midspan bending, gr1a				**Eqn. 6.10**			
Action		Symbol	Characteristic value (kNm)	γ factor	ψ factor	Load group factor	ULS design effect (kNm)
Dead load (steel)		$G_{\mathrm{k,DLst}}$	216	1.2	-	-	259
Dead load (concrete)		$G_{\mathrm{k,DLconc}}$	1218	1.35	-	-	1644
Superimposed dead load		$G_{\mathrm{k,SDL}}$	906	1.2	-	-	1087
gr1a	LM1 (TS)	$Q_{\mathrm{k,TS}}$	2955	1.35	1.0	1.0	3989
	LM1 (UDL)	$Q_{\mathrm{k,UDL}}$	1378	1.35	1.0	1.0	1860
	Pedestrian	$Q_{\mathrm{k,Ped}}$	82	1.35	1.0	0.6	66
Wind		$Q_{\mathrm{k,W}}$	51	Not considered with thermal			-
Thermal		$Q_{\mathrm{k,Th}}$	215	1.55	0.6	-	200
Total							9107

Table 5. ULS midspan bending with traffic group gr1a

ULS midspan bending, traffic group gr5

Load group gr5 consists of the frequent value of LM1 and the characteristic value of LM3, from the UK NA to BS EN 1991-2: 2003, **Table NA.3**:

$$Q_{\mathrm{k,gr5}} = \psi_1 Q_{\mathrm{k,LM1}} + Q_{\mathrm{k,LM3}} \tag{6}$$

Individual values of ψ_1 are defined for the different components of Load Model LM1 so these are applied individually:

$$Q_{\mathrm{k,gr5}} = \psi_{1,\mathrm{TS}} Q_{\mathrm{k,TS}} + \psi_{1,\mathrm{UDL}} Q_{\mathrm{k,UDL}} + Q_{\mathrm{k,LM3}} \tag{7}$$

Values of ψ_1 are found from the UK NA to BS EN 1990: 2002, **Table NA.A2.1**. The required values are those that correspond to the Load Model 1 components, hence values of 0.75 for the Tandem System and 0.75 for the UDL apply. The Note to Table NA.A2.1 clarifies that the ψ_1 factors given always apply to individual components of loading and the values for a given component are the same in all load groups in which the component occurs. Hence, although these loads are being combined in load group gr5, the ψ_1 values used are taken from the rows corresponding to the Tandem System and UDL which are defined under load group gr1a. Putting these ψ_1 factors into Equation (7) gives:

$$Q_{\mathrm{k,gr5}} = 0.75Q_{\mathrm{k,TS}} + 0.75Q_{\mathrm{k,UDL}} + Q_{\mathrm{k,LM3}} \tag{8}$$

The entire load group is applied as the leading variable action so substituting $Q_{\mathrm{k,gr5}}$ from Equation (8) for $Q_{\mathrm{k,Tr}}$ in Equation (3) gives:

$$E_{\mathrm{d,gr1a}} = E\left\{\begin{array}{l} 1.2G_{\mathrm{k,DLst}} + 1.35G_{\mathrm{k,DLconc}} + 1.2G_{\mathrm{k,SDL}} + \\ \quad 1.35\left[0.75Q_{\mathrm{k,TS}} + 0.75Q_{\mathrm{k,UDL}} + Q_{\mathrm{k,LM3}}\right] + 1.55 \cdot 0.6 \cdot Q_{\mathrm{k,Th}} \end{array}\right\} \tag{9}$$

Inserting the characteristic values from Table 2 into Equation (9) gives the result shown in Table 6:

ULS midspan bending, gr5				**Eqn. 6.10**			
Action		Symbol	Characteristic value (kNm)	γ factor	ψ factor	Load group factor	ULS design effect (kNm)
Dead load (steel)		$G_{\mathrm{k,DLst}}$	216	1.2	-	-	259
Dead load (concrete)		$G_{\mathrm{k,DLconc}}$	1218	1.35	-	-	1644
Superimposed dead load		$G_{\mathrm{k,SDL}}$	906	1.2	-	-	1087
gr5	LM1 (TS)	$Q_{\mathrm{k,TS}}$	341	1.35	1.0	0.75	345
	LM1 (UDL)	$Q_{\mathrm{k,UDL}}$	153	1.35	1.0	0.75	155
	LM3	$Q_{\mathrm{k,LM3}}$	4154	1.35	1.0	1.0	5608
Wind		$Q_{\mathrm{k,W}}$	51	Not considered with thermal			-
Thermal		$Q_{\mathrm{k,Th}}$	215	1.55	0.6	-	200
Total							9299

Table 6. ULS midspan bending with traffic group gr5

Comparison between Table 5 and Table 6 shows that load group gr5 gives the higher value to be used in design of the member (9299 kNm for gr5 compared with 9107 kNm for gr1a).

Note that the characteristic values used for LM1 in Table 6 are different to those used in Table 5. This is due to the need to combine the LM1 loading with the LM3 loading, in accordance with the combination rules given in the UK NA to BS EN 1991-2: 2003, **NA.2.16.4**. The characteristic values are the bending moment generated on a particular main beam by the applied loading. For gr1a, the lane 1 tandem system of LM1 is applied directly over the beam in question to generate the maximum moment. For gr5, the SV or SOV vehicle of LM3 is positioned directly over the beam, and the lane 1 tandem system of LM1 is applied to an adjacent lane, with the proportion of the load effect experienced by the beam depending on the transverse distribution by the deck of the adjacent lane of LM1.

Illustration 2: SLS Stress Check of Main Beam at Midspan

This illustration considers:

- SLS characteristic combination
- Differences between ULS and SLS combination equations

Basis of design

The steel stress check uses the characteristic combination, as directed by BS EN 1994-2: 2005, **7.2.2(5)** and BS EN 1993-2: 2006, **7.3(1)**, hence Equation 6.14b is used:

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq 1} G_{\mathrm{k},j} \text{ "+" } P \text{ "+" } Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1} \psi_{0,i} Q_{\mathrm{k},i}\right\}$$

EN1990 (6.14b)

The form of this equation is very similar to Equation 6.10 used for the ULS check above, except that there are no γ partial factors on the actions. As with the previous illustration, traffic loading is the leading action and thermal action is the only accompanying variable action. Replacing the summations with the appropriate actions from Table 2:

$$E_{\mathrm{d}} = E\{G_{\mathrm{k,DLst}} + G_{\mathrm{k,DLconc}} + G_{\mathrm{k,SDL}} + Q_{\mathrm{k,Tr}} + \psi_{0,\mathrm{Th}} Q_{\mathrm{k,Th}}\} \qquad (10)$$

From the comparison performed above for ULS bending it is apparent that load group gr5 generates greater sag moments than gr1a, hence gr5 will be used for this SLS check. Substituting the expression for $Q_{\mathrm{k,gr5}}$ from Equation (8) for $Q_{\mathrm{k,Tr}}$ in Equation (10) gives:

$$E_{\mathrm{d,gr1a}} = E\left\{\begin{matrix} G_{\mathrm{k,DLst}} + G_{\mathrm{k,DLconc}} + G_{\mathrm{k,SDL}} + \\ \left[0.75Q_{\mathrm{k,TS}} + 0.75Q_{\mathrm{k,UDL}} + Q_{\mathrm{k,LM3}}\right] + 0.6Q_{\mathrm{k,Th}} \end{matrix}\right\} \qquad (11)$$

Table 7 shows the result of this calculation. Applying the ψ factors to the characteristic values gives the design values of SLS bending moments. The resultant design moments are applied to the relevant cross section (steel alone, long term composite or short term composite) to obtain, in this example, the bottom flange stresses.

Shrinkage and thermal effects induce strains which would cause curvature of the section, if unrestrained, and residual stresses. For this statically indeterminate structure, secondary effects arise due to compatibility at the supports. The secondary moments are applied to the cross section to obtain secondary stresses, which are added to the residual primary stresses to obtain overall stress due to the shrinkage or thermal effect. The ψ factor applies to both the primary and secondary components.

SLS midspan moments/stresses, gr5			Eqn. 6.14b					
Action	Symbol	Char. value (kNm)	γ factor	ψ factor	Load group factor	SLS design effect (kNm)	Applied to section	Bottom flange stress (N/mm^2)
Dead (steel)	$G_{k,DLst}$	216	1.0	-	-	216	Steel	9
Dead (concrete)	$G_{k,DLconc}$	1218	1.0	-	-	1218	Steel	53
Superimposed DL	$G_{k,SDL}$	906	1.0	-	-	906	Long	31
Shrinkage		Not adverse at midspan					comp.	-
LM1 (TS)	$Q_{k,TS}$	341	1.0	1.0	0.75	256	Short	8
LM1 (UDL)	$Q_{k,UDL}$	153	1.0	1.0	0.75	115	comp.	4
LM3	$Q_{k,LM3}$	4154	1.0	1.0	1.0	4154		136
Wind	$Q_{k,W}$	51	Not with thermal					-
Thermal (2^{ndary})	$Q_{k,Th}$	215	1.0	0.6	-	129		4
(Primary)		stress	1.0	0.6	-	stress		-4
Total								243

Table 7. SLS midspan bending moments and bottom flange stresses

Illustration 3: ULS Check on Pier

This illustration considers:

- Transverse loads from traffic load groups
- Combination value for traffic load groups (using ψ_0)
- Partial factors for favourable and unfavourable effects

Basis of design

The centre pier is designed for the moments and shears caused by transverse loads (applied directly or through the bearing), together with the coexisting axial force due to vertical loads from the deck.

In this example, wind and traffic loads cause transverse forces to be applied through the bearing. The pier is also subject to accidental impact due to vehicle collision, which will be considered further in the next illustration. None of the permanent actions give rise to any transverse bending moments in the pier.

The effects caused by wind and traffic are treated as a persistent design situation, so Equation 6.10 is used at ULS:

$$E_{\mathrm{d}} = E\left\{\sum_{j\geq 1}\gamma_{\mathrm{G},j}G_{\mathrm{k},j} \text{ "+" } \gamma_{\mathrm{P}}P \text{ "+" } \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} \text{ "+" } \sum_{i>1}\gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i}\right\}$$

EN1990 (6.10)

The pier resistance check is an STR check, hence Set B factors, shown in Table 4, are used as before. It may be necessary to consider the envelope of minimum and maximum axial forces which can coexist with the transverse bending, hence calculations will be carried out using the unfavourable and favourable values of the partial factors.

In this example, the magnitudes of transverse forces on the pier due to wind and traffic shown in Table 2 are fairly similar (236 kN and 243 kN respectively). It is not immediately possible to tell by inspection which will govern as the leading variable action, so each will be tried in turn.

Transverse forces are generated by traffic load groups gr2 and gr6 (see Table 3 and the UK NA to BS EN 1991-2: 2003, **Table NA.3**). Of these, gr2 has the transverse forces associated with Load Model LM1 and gr6 has transverse forces associated with Load Model LM3. Table 2 shows that gr6 produces higher transverse forces in this example (243 kN compared with 226 kN), hence gr6 will be used.

Note that there is a limiting pressure applied to wind actions when applied in combination with traffic actions. See the UK NA to BS EN 1991-1-4: 2005, **NA.2.44**, and the combination rules given in BS EN 1990: 2002, **A2.2.2(5)**.

Transverse pier behaviour, combination value of traffic (using ψ_0)

Consider the transverse behaviour of the pier, with wind as the leading variable action and traffic accompanying. Inserting the relevant terms and partial factors into Equation 6.10 gives Equation (12). Note that the permanent loads make no transverse contribution so this term drops out.

$$E_{\mathrm{d}} = E\left\{\left[\sum_{j\geq 1}\gamma_{\mathrm{G},j}G_{\mathrm{k},j}\right] + 1.7Q_{\mathrm{k,W}} + 1.35\psi_{0,\mathrm{Tr}}Q_{\mathrm{k,Tr}}\right\} \tag{12}$$

Traffic is the accompanying action, hence the ψ_0 value is required from the UK NA to BS EN 1990: 2002, **Table NA.A2.1**. This Table shows that ψ_0 is zero for traffic load group gr6 and indeed all load groups other than gr1a, i.e., only load group gr1a ever needs to be considered when traffic is an accompanying action. Note also that gr1a does not contain any transverse load components. Hence, road traffic only contributes to transverse or longitudinal loading when it is the leading action. Inserting the ψ_0 value into Equation (12) gives:

$$E_{\mathrm{d}} = E\left\{1.7Q_{\mathrm{k,W(transv)}} + 1.35\cdot 0\cdot Q_{\mathrm{k,Tr(transv)}}\right\} \tag{13}$$

Inserting the characteristic values for transverse forces from Table 2 into Equation (13) gives the result shown in Table 8:

ULS transverse force on pier, wind leading			**Eqn. 6.10**			
Action	Symbol	Characteristic value (kN)	γ factor	ψ factor	Load group factor	ULS design effect (kN)
Dead load (steel)	$G_{k,DLst}$	No transverse component	-	-	-	-
Dead load (concrete)	$G_{k,DLconc}$		-	-	-	-
Superimposed dead load	$G_{k,SDL}$		-	-	-	-
gr6 LM3 (vertical)	$Q_{k,LM3}$	No transverse component	-	-	-	-
gr6 LM3 (transverse)	$Q_{k,Tr(transv)}$	243	1.35	0	1.0	0
Wind	$Q_{k,W}$	236	1.7	1.0	-	402
Thermal	$Q_{k,Th}$	No transverse component	Not considered with wind			-
Total						402
Applying the transverse force to the top of the pier gives a maximum pier bending moment of 2613 kNm and shear of 402 kN.						

Table 8. ULS transverse forces acting at top of pier, wind leading

Transverse pier behaviour, traffic leading

Consider traffic as the leading variable action. No ψ factor is therefore applied to it.

For accompanying wind actions in persistent design situations, a value of $\psi_0 = 0.5$ is obtained from UK NA to BS EN 1990: 2002, **Table NA.A2.1**. Inserting into Equation (12) gives:

$$E_d = E\{1.35 Q_{k,Tr(transv)} + 1.7 \cdot 0.5 \cdot Q_{k,W(transv)}\} \tag{14}$$

Inserting the characteristic values for transverse forces from Table 2 into Equation (14) gives the result shown in Table 9:

ULS transverse force on pier, traffic gr6 leading				**Eqn. 6.10**			
Action		Symbol	Characteristic value (kN)	γ factor	ψ factor	Load group factor	ULS design effect (kN)
Dead load (steel)		$G_{k,DLst}$	No transverse component	-	-	-	-
Dead load (concrete)		$G_{k,DLconc}$		-	-	-	-
Superimposed dead load		$G_{k,SDL}$		-	-	-	-
gr6	LM3 (vertical)	$Q_{k,LM3}$	No transverse component	-	-	-	-
	LM3 (transverse)	$Q_{k,Tr(transv)}$	243	1.35	1.0	1.0	328
Wind		$Q_{k,W}$	236	1.7	0.5	-	201
Thermal		$Q_{k,Th}$	No transverse component	Not considered with wind			-
Total							529
Applying the transverse force to the top of the pier gives a maximum pier bending moment of 3439 kNm and shear of 529 kN.							

Table 9. ULS transverse forces acting at top of pier, traffic gr6 leading

Comparison between Table 8 and Table 9 shows that traffic as the leading variable action gives the higher transverse force on the pier (529 kN compared to 402 kN), and hence higher bending moments and shears.

Maximum coexisting vertical loads on pier

Consider the coexisting axial force in the pier for the bending and shear produced when traffic is the leading variable action. Maximum and minimum coexisting axial forces will be determined.

The maximum axial force will be generated by positioning the LM3 vehicle directly over the pier and using the unfavourable partial factors, $\gamma_{G,sup}$, from Table 3. Updating Equations (12) and (14) to include all applicable vertical loads gives:

$$E_{d,gr6(vert)} = E\{1.2G_{k,DLst} + 1.35G_{k,DLconc} + 1.2G_{k,SDL} + 1.35Q_{k,LM3} + 1.7 \cdot 0.5 \cdot Q_{k,W}\} \quad (15)$$

Inserting the characteristic values for vertical pier reactions from Table 2 into Equation (15) gives the result shown in Table 10:

ULS axial force on pier (maximum)				Eqn. 6.10			
Action		Symbol	Characteristic value (kN)	γ factor	ψ factor	Load group factor	ULS design effect (kN)
Dead load (steel)		$G_{k,DLst}$	138	1.2	-	-	166
Dead load (concrete)		$G_{k,DLconc}$	777	1.35	-	-	1049
Superimposed dead load		$G_{k,SDL}$	577	1.2	-	-	692
gr6	LM3 (vertical)	$Q_{k,LM3}$	1642	1.35	1.0	1.0	2217
	LM3 (transverse)	$Q_{k,Tr(transv)}$	No vertical component	-	-	-	-
Wind		$Q_{k,W}$	24	1.7	0.5	-	20
Thermal		$Q_{k,Th}$	30	Not considered with wind			-
Total							4144

Table 10. Maximum coexisting ULS axial force on pier

Minimum coexisting vertical loads on pier

The minimum coexisting axial force will be generated using the favourable values of partial factors, $\gamma_{G,inf}$, and positioning the LM3 vehicle away from the pier. Note that these axial forces coexist with the transverse forces shown in Table 9, hence the same partial factors must be used for traffic and wind actions. Revising Equation (15) with the lower partial factors gives:

$$E_{d,gr6(vert)} = E\{0.95G_{k,DLst} + 0.95G_{k,DLconc} + 0.95G_{k,SDL} + 1.35Q_{k,LM3} + 1.7 \cdot 0.5 \cdot Q_{k,W}\} \quad (16)$$

Inserting the characteristic values for vertical pier reactions from Table 2 into Equation (16) gives the result shown in Table 11:

ULS axial force on pier (minimum)				Eqn. 6.10			
Action		Symbol	Characteristic value (kN)	γ factor	ψ factor	Load group factor	ULS design effect (kN)
Dead load (steel)		$G_{k,DLst}$	138	0.95	-	-	131
Dead load (concrete)		$G_{k,DLconc}$	777	0.95	-	-	738
Superimposed dead load		$G_{k,SDL}$	577	0.95	-	-	548
gr6	LM3 (vertical)	$Q_{k,LM3}$	1167	1.35	1.0	1.0	1575
	LM3 (transverse)	$Q_{k,Tr(transv)}$	No vertical component	-	-	-	-
Wind		$Q_{k,W}$	24	1.7	0.5	-	20
Thermal		$Q_{k,Th}$	30	Not considered with wind			-
Total							3013
So pier is checked for combined forces: Bending: 3493 kNm (from Table 9) Shear: 529k N (from Table 9) Axial: Envelope of minimum 3013 kN to maximum 4144 kN (from Tables 10 and 11)							

Table 11. Minimum coexisting ULS axial force on pier

Note that the absolute minimum vertical force on the pier may be required for some checks, such as bearing design, in which case no traffic load would be applied. In this example, vertical traffic loading is required to generate the coexisting transverse force.

Illustration 4: ULS Check on Pier for Accidental Impact

This illustration considers:

- Combination equation for accidental design situations
- Frequent value for traffic load groups (using ψ_1)

Accidental situation

The accidental design situation of vehicle impact on the pier will be considered, using Equation 6.11b:

$$E_d = E\left\{\sum_{j\geq 1} G_{k,j} \text{ "+" } P \text{ "+" } A_d \text{ "+" } (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{k,i}\right\}$$

EN1990 (6.11b)

The centre pier is situated beside a road under the bridge. Due to space constraints it is not possible to install high containment barriers in accordance with the recommendations of PD 6688-1-7: 2009, **2.1.1**[2]. The pier therefore needs to be designed to withstand vehicle impact forces in accordance with the UK NA to BS EN 1991-1-7: 2006, **NA.2.11.2**.

The equivalent static design forces for the accidental impact are shown in Table 2 for the main and residual components. These act parallel to the direction of travel, i.e., horizontally with respect to the pier, and are distributed to the deck and substructure by the pier acting in bending and shear.

Take traffic as the leading variable action and thermal as the accompanying variable action. BS EN 1990: 2002, **A2.2.5(2),** confirms that traffic loads on the bridge deck should be taken into account using their frequent value for an accidental design situation involving impact from traffic under the bridge. Clause **A2.2.5(1)** confirms that wind actions do not need to be considered together with accidental actions.

The frequent values of traffic load groups have a different definition to the characteristic values used in the previous illustrations. A subset of three of the load groups is defined at the frequent value by BS EN 1991-2: 2003, **Table 4.4b**, consisting of single components of LM1 (gr1a), LM2 (gr1b) or pedestrian load (gr3). Frequent values for other load groups are not required to be considered.

Transverse pier behaviour under impact load

The frequent values of the traffic load groups defined in BS EN 1991-2: 2003, **Table 4.4b** consist of vertical components only and do not contain any transverse components. The thermal action does not include any transverse component. The permanent loads do not include any transverse components. Therefore most of the terms drop out of Equation 6.11b for transverse loads:

$$E_{\mathrm{d,transv}} = E\left\{\left[\sum_{j\geq 1} G_{\mathrm{k},j}\right] + A_{\mathrm{d}} + \left[\psi_{1,1} Q_{\mathrm{k,Tr}}\right] + \left[0.5 Q_{\mathrm{k,Th}}\right]\right\} \tag{17}$$

Inserting the values for horizontal pier impact forces from Table 2 into Equation (17) gives the result shown in Table 12:

ULS transverse force on pier, accidental situation				Eqn. 6.11b			
Action			Characteristic value (kN)	γ factor	ψ factor	Load group factor	ULS design effect (kN)
Dead load (steel)		$G_{\mathrm{k,DLst}}$	No transverse components	-	-	-	-
Dead load (concrete)		$G_{\mathrm{k,DLconc}}$		-	-	-	-
Superimposed dead load		$G_{\mathrm{k,SDL}}$		-	-	-	-
Accidental		A_{d}	1500 750	1.0 1.0	1.0 1.0	- -	1500 750
gr1a	LM1 TS	$Q_{\mathrm{k,TS}}$	No transverse components	-	-	-	-
	LM1 UDL	$Q_{\mathrm{k,UDL}}$		-	-	-	-
Thermal		$Q_{\mathrm{k,Th}}$		-	-	-	-

The main component of 1500 kN is applied to the pier at the most severe point between 0.75m and 1.5m above carriageway level, giving a ULS moment of 1864 kNm at the base of the pier. The residual component of 750 kN is applied to the pier at the most severe point between 1m and 3m above carriageway level, giving a ULS moment of 938 kNm at the base of the pier. Design effects at the base of the pier due to this accidental action are therefore:

Bending: 2802 kNm
Shear: 2250 kN

Table 12. ULS transverse forces due to pier impact in accidental design situation

Axial loads on pier coexisting with impact loading

Updating Equation (17) to consider the coexisting vertical loads from the deck gives:

$$E_{\mathrm{d,vert}} = E\{G_{\mathrm{k,DLst}} + G_{\mathrm{k,DLconc}} + G_{\mathrm{k,SDL}} + A_{\mathrm{d}} + \psi_{1,\mathrm{Tr}}Q_{\mathrm{k,Tr}} + \psi_{2,\mathrm{Th}}Q_{\mathrm{k,Th}}\} \tag{18}$$

Load model LM3 is not considered at the frequent value (see BS EN 1991-2: 2003, **Table 4.4b**). Therefore the maximum vertical force will be generated using load group gr1a, with the tandem system placed over the pier. Note in comparison with Equation (4) that there is no pedestrian component to gr1a at the frequent value:

$$\psi_{1,\mathrm{Tr}}Q_{\mathrm{k,gr1a}} = \psi_{1,\mathrm{TS}}Q_{\mathrm{k,TS}} + \psi_{1,\mathrm{UDL}}Q_{\mathrm{k,UDL}} \tag{19}$$

The frequent value of the traffic load groups is being used here in the combination equation for accidental design situations. The frequent value of traffic loads is also used in the SLS frequent combination, Equation 6.15b. This combination is used, for example, for crack control verifications for prestressed concrete members, see BS EN 1992-2: 2005, **Table 7.101N**, hence load model LM3 is not considered in such verifications.

Values of ψ_1 are found from the UK NA to BS EN 1990: 2002, **Table NA.A2.1**, hence values of 0.75 for the Tandem System and 0.75 for the UDL apply. Note that these are the same ψ_1 values, taken from the same place in the table, that were used for the frequent value of load model LM1 as part of load group gr5, in Equation (8).

For thermal actions, a value of $\psi_2 = 0.5$ is obtained from UK NA to BS EN 1990: 2002, **Table NA.A2.1**.

Substituting $Q_{\mathrm{k,gr1a}}$ from Equation (19) for $Q_{\mathrm{k,Tr}}$ in Equation (18) gives:

$$E_{\mathrm{d}} = E\left\{\begin{array}{l} G_{\mathrm{k,DLst}} + G_{\mathrm{k,DLconc}} + G_{\mathrm{k,SDL}} + \\ \quad A_{\mathrm{d}} + [0.75Q_{\mathrm{k,TS}} + 0.75Q_{\mathrm{k,UDL}}] + 0.5Q_{\mathrm{k,Th}} \end{array}\right\} \tag{20}$$

Inserting the characteristic values for vertical pier reactions from Table 2 into Equation (20) gives the result shown in Table 13.

ULS axial force on pier, accidental situation				**Eqn. 6.11b**			
Action		Symbol	Characteristic value (kN)	γ factor	ψ factor	Load group factor	ULS design effect (kN)
Dead load (steel)		$G_{k,DLst}$	138	1.0	-	-	138
Dead load (concrete)		$G_{k,DLconc}$	777	1.0	-	-	777
Superimposed dead load		$G_{k,SDL}$	577	1.0	-	-	577
Accidental		A_d	No vertical component	-	-	-	-
gr1a freq.	LM1 TS	$Q_{k,TS}$	595	1.0	1.0	0.75	446
	LM1 UDL	$Q_{k,UDL}$	712	1.0	1.0	0.75	534
Thermal		$Q_{k,Th}$	30	1.0	0.5	-	15
Total							2487

The minimum axial force can be found by omitting all the variable loads from the deck, i.e., minimum axial force is 1492 kN.
This axial force needs to be combined with the other design effects, i.e., pier is checked for:
Bending: 2802 kNm (from Table 11)
Shear: 2250 kN (from Table 11)
Axial: Envelope of minimum 1492 kN to maximum 2487 kN (from Table 12)

Table 13. ULS axial force on pier coexisting with pier impact in accidental design situation

SESSION 1-4:

EN 1992 – CONCRETE

Bridge Design to Eurocodes: UK Implementation
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THE UK NA FOR EN 1992-2

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Abstract
The background to key provisions of the UK National Annex (NA) to EN 1992-2 is discussed. Areas considered include web crushing. For this, the UK NA makes EN 1992 slightly more conservative for some cases and significantly more conservative for some more unusual cases where it was felt there was insufficient justification for major relaxations of past practice. Short shear span enhancement is also considered where the UK changed the rules mainly for ease of use although it sometimes makes design more economic. The recommended approach and values for cracking criteria are largely followed even though they represent a significant change to past practice as the justification.

Introduction
Some parts of Eurocodes, notably the Environmental Actions sections of EN 1991, have National Determination that is due to genuine physical differences between countries. There is also National Determination due to the fact that safety remains a National responsibility. However, very few of the NDPs or other areas where National Determination is allowed in the material sections are really due to differences between Nations except in dominant opinion. Most of them were provided because when it was not possible to reach full agreement, it was decided to allow National Determination so that all countries could do something they were happy with. Consequently, as in most parts of the Eurocodes, the sections of EN 1992-2 which were most controversial can be identified by the number of Nationally Determined Parameters (NDPs). Naturally, it is the countries that did not get their favoured solutions adopted that are most inclined to depart from the recommended method or values. So it is that the UK generally adopts Recommend Values and it is mainly where the UK tried but failed to get significant changes that we depart from these. There are also some places where allowance for National Determination allowed us to include explanation and occasionally unrelated changes which were considered beneficial.

Various factors in the UK NA to EN 1992-2 are considered in this paper. Some of the more major decisions have been the subject of papers which will be referred to and accordingly space is given to smaller changes which have not previously been explained as fully.

Some of the NDPs used in EN 1992-2 are common with EN 1992-1. The background to these is given in the PD 6687-1 and is not discussed here with the exception of the web crushing limits where the issue is more often critical to bridges and the work was done as part of the calibration of EN 1992-2. Further background to the EN 1992-2 NA is given in PD 6687-2.

Structure of EN 1992-2
All the materials Eurocodes have the same basic structure for their initial 7 sections, starting with Introduction followed by Basis of Design in Section 2, Materials in Section 3, Durability

in Section 4, Analysis in Section 5, Ultimate Limit States in Section 6, Serviceability in Section 7. The material is also divided by effect rather than element type. Thus for example, in Sections 6.1 and 6.2 of EN1992-2 respectively, the flexural and shear resistance with or without axial force is considered in contrast with BS 5400 which had separate sections on beams, slabs and columns.

This layout of sections is the same as for all materials but is a bigger departure for concrete as traditionally reinforced and prestressed concrete were considered in separate clauses or even completely separate codes. The layout does have some practical advantages, avoiding repetition, reducing references between sections and automatically covering, for example, the effect of axial forces on beams and slabs.

The integration and consistent treatment of reinforced and prestressed is not as total as some would have liked and significant differences, particularly in serviceability criteria, do remain as will be seen later in this paper.

Calibration of EN 1992-2

Some calibration studies of EN 1992-2 and before that ENV 1992-2 were done in the UK, notably for the Highways Agency. The calibration was generally against BS 5400 and where results were close the EN was considered acceptable. However, where significant differences between the codes were found, test data and other evidence were reviewed to see whether the change was justified. Some noticeable differences were found and this work informed decisions on the National Annex. In some cases (including in upper limits in shear and cracking criteria which will be considered below) it was concluded that significant changes from BS 5400 were justified or, in other words, that EN 1992 represented a significant improvement.

Shear

There are two major areas where the UK NA departs from recommended values in shear as discussed below, *viz* short shear span enhancement and the upper web crushing limit.

Short shear span enhancement

Quite late in the drafting of EN 1992-1-1, it was decided to change the short shear span approach given in EN1992-1-1, **6.2.2(1)** from enhancing resistance for short shear spans as in BS 5400 to reducing the load considered when the load is near the support. The UK team working on EN 1992-2 identified this as a practical problem for bridge designers. Because of the multiplicity of load cases which have to be considered in bridge design (even with EN 1991-2 highway loading which is considerably simpler than the previous BS loading) it is common practice to design for envelopes of load cases, rather than individual load cases. This is not straightforward with the reduced load approach to short shear span enhancement. There is also a more general problem of inconvenience as the shear force that has to be designed for does not match anything that comes out of any normal form of analysis. It would be possible to reduce the load in the analysis but this would require a different loading to be applied for considering short shear span for each section it was considered for.

The reduced load approach also gives less short shear span enhancement when multiple loads are considered as the reduction depends on the position of each individual load, rather than of the section considered in shear. This means the reduced load approach is considerably more conservative (as well as more difficult to use) for (for example) uniformly distributed loads than the increased resistance approach. This was considered correct by some countries. However, this is rather strange as the previous EN increased resistance approach had been calibrated for uniformly distributed loads which resulted in reducing the enhancement from being based on 2.5d to 2.0d and this change was not reversed when the approach was changed. After much discussion, the BSi bridges group decided to resolve the problems by what they hoped was a short term measure pending later discussion with the relevant CEN committee, SC2. The short shear span clauses are not subject to national determination but the normal shear strength without links rules are. The UK NA to EN 1992-2 therefore reinstates the enhanced resistance approach by inserting it in the normal shear clause. There is a very important proviso, to avoid double counting the benefit, that this can only be used if the reduced load approach used by EN 1992 is not used as well.

The subject of short shear span enhancement is covered in a paper by Jackson et al[1]. This also notes that the reduced load approach could give odd results if applied with loads in both directions as well as raising further issues for sections with links. The UK view is that EN 1992 short shear span approach needs significant review. This has been raised in SC2 (the committee responsible for EN 1992) but has not yet achieved sufficient support to be acted on. This is an example of the diversity of views across Europe. The UK view is that the present approach is a practical problem and also that in reality short shear span effect must increase resistance, rather than reduce shear force as the force is controlled by the laws of physics.

Upper (web crushing) limit

EN 1992 gives a significant increase in the upper or web crushing limit to shear strength compared with BS 5400. Although it was known that the BS 5400 figures were conservative, it was felt the increase was so large it should be investigated. No convincing evidence the rules (given in EN1992, **6.2.3**) were unsafe was found. However, some concerns were raised which led to some departures from recommended values for related National Determined Parameters. These are discussed in a paper by Jackson and Salim[2] but will be reviewed briefly here.

Although the UK changes make the rules slightly more conservative (and significantly more conservative for some extreme and unusual cases), the web crushing limit is still significantly higher than in BS 5400. This will be a significant benefit for some types of structures, notably box girder bridges. Thinner webs over the supports may simplify formwork avoiding the need for varying thickness and also give a lighter, more efficient section needing less prestress.

Making full use of the maximum shear stress allowed by EN 1992-2 does require a lot of links and will rarely be beneficial as the gain in shear resistance with link area rapidly diminishes as the maximum is approached as will be seen from Figure 1. However, it does give a less obvious advantage for designers. The optimum design for a box girder to BS 5400 would have webs on the minimum thickness, i.e. at point A on Figure 1. However, there is no direct way of determining the required thickness exactly. It would have to be estimated. The prestress design and stress checks are then completed and the web design checked. If the stress is slightly higher than estimated, there is no solution except to increase web thickness. If concrete strength is already 60 (cube) as will often be the case, the option of increasing concrete strength will not help either as BS 5400 gave no benefit. It will therefore be necessary to redo the prestress design calculations for the new section. An optimum design to EN 1992, in contrast, would have thinner webs perhaps in the region of B on Figure 1. However, if detail design calculations show the shear stress to be slightly greater than anticipated, the amount of shear reinforcement can simply be increased.

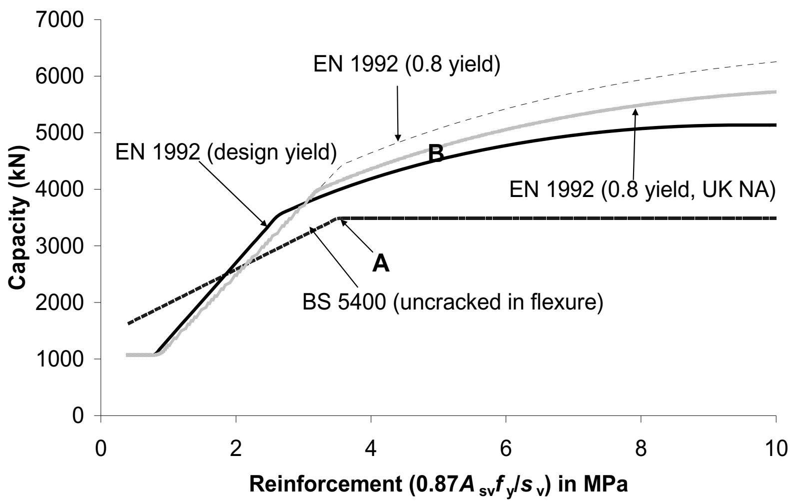

Figure 1: Link design rules and web crushing limit compared
(50/60 Concrete, b=300 d=1900, h = 2000, 7MPa axial stress, vertical links)

80% yield rule

The web crushing stress in EN 1992 can be increased if the stress in the links used in link design is limited to 80% of yield strength. Because of the way the EN 1992 varying angle truss approach works, this not only increases the absolute maximum shear that can be taken regardless of link area but also (by allowing a flatter truss) increases strength with somewhat fewer links as is apparent from Figure 1. Fewer links may be required if the links are designed assuming they are stressed to 80% yield rather than fully stressed to design strength of characteristic yield/γ_s = 87% yield. This is so counter-intuitive many users of the code (and indeed advisory publications) have not taken advantage of the rule. It is also hard to explain physically particularly when a range of steel grades are considered. It appears to imply that increasing steel yield strength can increase member strength even when the stress and strain state is totally unchanged by this. Because the 80% rule has no gamma factor, it is not obvious how to calibrate it against tests. However, it did appear some tests were excessively near the limits and it was decided to make the rule marginally more conservative in the UK NA.

Slender webs and inclined links

The varying angle truss analogy can give significantly higher web crushing limits with inclined links. This is correct in theory but there do not appear to be any tests with enough inclined links to prove it. Enquiries to several countries elicited the response “we do not have any test with inclined links but it does not matter because we never use them”. This may be true, but they never previously had any incentive to use them whilst the introduction of EN1992 gives such an incentive so it seemed a rather dubious argument and anyway if you never use inclined links removing their advantage in the code would not give any disadvantage.

The Eurocode also does not take any account of depth to thickness ratio. The possibility that concrete webs might be slender enough to have buckling issues like steel webs was considered but it appeared they were not slender enough for this. However, a statistical analysis by Batchelor et al[3] suggests web depth to thickness ratio is a significant factor in web crushing stress. The statistical analysis was found on closer examination to be slightly dubious in this respect. Nevertheless there was serious concern as EN 1992 has no limit on web slenderness whilst the most slender webs found in practice, such as on the original A2/M2 Medway Bridge, are up to some four times more slender than any for which there are published test data available.

BS 5400 does not consider slenderness either but as its web crushing limit is much more conservative this is less of a concern. It is also understood a model of the original A2/M2 Medway Bridge (which had stresses similar to the BS 5400 limits) was tested although this was never published.

After considering these issues, it was decided to make the crushing limit with link stress of $0.8f_y$ slightly more conservative in the UK NA than the recommended value. As the decision was made because of lack of evidence, rather than because of evidence the recommended values were unsafe, the restrictions put in the UK NA inevitably involved a degree of judgement. It is not anticipated they will often have any effect. As noted earlier, inclined links are rarely used whilst very slender webs do not often arise. A modern version of the Medway Bridge would be likely to have fewer webs and a deck slab with a longer transverse span as indeed the new bridge, which has been constructed adjacent to it for widening, has.

Cracking

A major change to the cracking section of 1992-2 was proposed very late in the drafting which would have introduced significant differences from 1992-1-1. Most countries, including some which would have preferred different cracking approaches including the UK, felt it preferable to maintain consistency with 1992-1-1 and the strong difference of view was eventually resolved by making the whole section subject in EN1992-2 open to national determination. The UK kept the “recommend approach” but made some minor changes to Table 7.101N defining criteria. We also kept the recommended criteria for crack widths in reinforced concrete but as they are significantly different from previous UK requirements they will be considered here.

Criteria for reinforced concrete

There is a significant relaxation in cracking criteria compared with BS 5400. These were subject to national determination but it was decided there was no justification for changing them. Crack widths in reinforced concrete are verified under the quasi-permanent combination of actions, which does not include any traffic actions (since $\psi_2 = 0$ for traffic actions, see EN1990). This approach is reasonable since cracking is a concern for aesthetic and durability reasons, thus it is the crack opening that occurs for extended periods of time rather than under short term loading that is of concern. This approach is only valid because the stress limits in EN1992 require that the reinforcement remains linear-elastic under the characteristic combination of actions, which does include traffic actions.

Furthermore, although it is traditionally assumed that there is some relationship between crack width and corrosion, not much evidence for this has been found despite extensive research going back many years[4]. It was therefore decided not to alter this requirement. Similarly it was decided not to introduce BS 5400's requirement for stricter crack width control with more severe environments as there appears to be no evidence to justify doing so.

Although the quasi-permanent combination used for the cracking check does not include any traffic load, it does include some temperature effect. This did not arise in BS 5400 where the "modified combination 1" was used. The temperature effect from EN 1991-1-5 and its National Annex includes a non-linear distribution (i.e. the temperature difference component). It is not at all clear what to do with the self equilibrating component of this and the UK NA confirms that it can be ignored in crack width checks, although not in "decompression" or stress checks.

Criteria for prestressed concrete

Although EN 1992 integrates the treatment of reinforced and prestressed concrete, it still requires more onerous cracking criteria for prestressed concrete ostensibly to protect the tendons from corrosion. There is, however, an important relaxation compared with BS 5400. The "decompression" check under the frequent combination of actions (roughly equivalent to BS 5400's class 1 under modified combination 1 check) is only required where you have chloride exposure. Again there did not appear to be much evidence for a relationship between cracking and corrosion. Indeed, it is now broadly accepted that cracks parallel to bars or tendons are more likely to affect their susceptibility to corrosion than those perpendicular. This implies that *if* cracking really does have to be avoided to give adequate corrosion protection to tendons, cracking in all directions should be prevented. Interestingly, EN 1992 does not explicitly say whether this applies. The definition of decompression is that concrete within a certain distance from the tendons stays in compression. It does not actually specify any particular direction and could be read as meaning all directions. However, BS 5400 has completely separate sections on reinforced and prestressed concrete and does not say anywhere that you can design structures or elements as prestressed in one direction only. Conventional practice says it can be done and it will continue to be done with EN 1992. It is, however, not entirely clear it is logical. Those who advocate partial prestressing will argue that EN 1992-2 has moved in the right direction by dropping the decompression check when you do not have chloride exposure but possibly not far enough and would argue that the next logical step is to drop the decompression check for all cases.

The apparent reason for the decompression restriction is to protect the tendons from corrosion such as due to chloride ingress. The recommend value of the distance from tendons where decompression has to be considered in EN 1992-1-1 is 25mm, and is increased to 100mm in EN 1992-2. The latter is usually more than the cover required for durability which appears illogical. If cover required for durability is 40mm, the code appears to imply that 40mm of concrete in compression provides adequate protection, but 40mm of concrete in compression plus another 60mm which might be in tension is somehow assumed to provide less protection. To avoid this odd implication the UK changed the distance to $c_{min,dur}$, the cover required for durability. Since there are normally links or other reinforcement outside the tendons and since nominal cover includes an allowance for deviation, the nominal cover will typically be 25mm or so more than $c_{min, dur}$. This means there is a small area near the tension face where decompression does not have to be checked. This might not appear significant but for a 220mm thick prestressed slab, it would increase the allowable load before decompression by some 30%. It seems unlikely, however, this will be sufficient to make transverse prestressing of bridge deck slabs popular.

The decompression criterion used with “XD” and XS” chloride exposure are applied only near tendons, whereas crack widths, which are checked with other exposure classes, are checked everywhere. Because it only states the requirement for the decompression check for XD and XS chloride exposure, the recommended table on crack widths, EN 1992-2: **Table 7.101N**, could be misinterpreted to mean that concrete away from the tendons has no check at all in such cases, whereas it would sensibly be expected to have a crack width check. A note was therefore added to the NA table to say crack width should be checked where decompression does not have to be checked. This is only a clarification not a change. In EN 206 if you have XD (chloride) exposure you also have XC (carbonation) exposure and therefore the crack width limit still applies. Similarly, in the table header, the NA has changed the requirement that sections with “unbonded or external prestress” are treated like RC to saying “sections without bonded prestress”. This is to clarify the position for structures with both bonded and unbonded prestress. The text of the EN does already do this, but the change in column heading should avoid the potential for misinterpretation.

There can be some confusion where the most tensile face of a section is not subject to such severe exposure as other faces, for example where the top of a cantilever slab is protected by waterproofing but the bottom is not. The NA also attempts to clarify this.

Despite the apparently radical EN 1992 approach of considering RC and prestressed together, the tighter cracking criteria for prestressed concrete have the effect that SLS criteria still tend to govern prestressed design and ultimate criteria RC design. As in many parts of the Eurocodes, it is not as radically different from past practice as it appears. Similarly, although in principle cracked section analysis is used at SLS for prestressed, in practice it is only where there is no chloride exposure (or no bonded tendons) that this really arises. This is due to the decompression limit and the fact that sections can be considered as uncracked provided stress does not exceed the effective tensile strength. This usually means that a section designed for decompression under frequent load can still be treated as uncracked under full characteristic load.

Calibration studies suggested that conventional structures subjected to chloride exposure come out very similar to EN 1992 and BS 5400. However, without chloride exposure, significant savings in prestress can be made.

Ductility

There are clauses in EN 1992-2 designed to ensure that if tendons corrode to such an extent that the structure was greatly weakened, any failure would always give warning and the structure would not fail as soon as it cracks. Whilst such robustness requirements are sensible, the UK was not wholly convinced of the actually philosophy included in EN1992. Indeed it appears in some cases providing additional secondary steel to comply with them would make loss of tendons less visible.

Alternative approaches are provided. The fundamental approach is to check the section with the reduced prestress and in most cases it appears the rules will not govern design. A simpler approach of providing a certain amount of secondary steel is also allowed which will reduce the amount of calculation work required. Pretensioned tendons can be included in this area provided they have more than a certain cover. The recommended value of this is twice the cover required for durability $c_{\mathrm{min,dur}}$. The UK changed this to $c_{\mathrm{min,dur}}$ so that all the tendons can be included and the criterion will not be critical.

In this and some other areas of EN 1992-2 the differing practices across Europe were evident. Some countries do not use pretensioning in bridges at all, whilst others use it extensively. The former were inclined to propose clauses which, whilst perfectly reasonable for post-tensioned structures, precluded what has been found to be perfectly reasonable practice in pretensioning.

Fatigue of Reinforcement

EN 1992 has rules on fatigue of reinforcement. Since many countries have not previously considered this, there was some discussion of whether simple bridges could be exempted. However, since fatigue is due to live load stress it is actually smaller structures which are *more* likely to be affected by it. EN 1992-2 does specifically say that some types of structure (notably footbridges and most substructures) do not have to be checked and allows the NA to add more. The UK has added most slabs of beam and slab type bridges to this. The reason this is justified is that in reality the real stress range is much lower than that calculated by conventional elastic analysis because of the effect of “compression membrane action”. Since deck slabs have both a high live load ratio and a high number of load cycles, they would otherwise be likely to be governed by fatigue so this is a significant relaxation. The rules are based on Canadian work. The Canadian standard has further restrictions but, provided these are complied with, goes much further in allowing only 0.3% reinforcement each way to be used in such slabs with no further design calculations.

Annexes

B creep and shrinkage

Additions to Annex B in EN 1992-2 compared with 1992-1 give a different creep model for high strength concrete, particularly that with silica fume. It operates in a similar fashion to the shrinkage model by splitting the strain into autogenous and drying components. It appears to have theoretical advantages. However, the UK were not convinced it was valid for higher relative humidities as it switches to swelling at implausibly low relative humidity values (well below 100%). This was raised with the drafters and Project Team resulting in the scope being restricted to cases with RH below 80%. However, the UK team were not convinced it was valid at this level and furthermore, UK structures are normally above RH 80% for significant periods of their life. It was therefore decided not to adopt this annex.

It is worth noting that the creep and shrinkage rules in EN 1992 are based on the 1990 CEB/FIP model code approach which is known to be a significant advance on the 1978 CEB/FIP model code used in the appendix to BS 5400. This effectively treated all creep and shrinkage as drying and so, for example, exaggerated the effect of element thickness on timescale. EN 1992 also uses formulae rather than diagrams making it easier to implement within spreadsheets and other computer software.

LL and MM membrane and shells

Annexes LL and MM provide an approach for considering membranes and shells. The rules, which will be considered in a later paper, can be used, for example, to check the walls of a box girder bridge using results from a conventional elastic finite element analysis. There is, however, a potential issue with this. It appears that if you designed such a structure for ultimate resistance near the limit using the normal provisions of Section 6, it would almost always fail if you checked using the membrane rules. This occurs because the membranes rules do not allow the same degree of redistribution round the section, although how much they do allow depends, for example, on whether you apply them to a whole web or individual smaller elements. Also, they have different sensitivities to some effects. In particular, whilst in the normal web crushing rules normal levels of longitudinal compression would increase maximum shear stress, in the membrane rules they would reduce it. After discussion in the CEN committees it was decided to put these in an informative annex and the UK NA does allow its use. Because of the differences from the normal shear rules, it appears that there is a potential issue with independent checks and designers would be well advised to specify the approach at Approval in Principal stage.

The main text of EN 1992-2 does explicitly say that interaction between out of plane moments and shears in webs can be ignored if either is below a certain level. However, once above that level they do have to be considered and the membrane and shell rules provide a way of doing this.

PP non linear safety factor format

As a result of the different material safety factor values and treatment for steel and concrete, the safety factor approach to be used with non-linear analysis of concrete structures is not obvious. EN 1992-2 specifies an approach. The description of this is not very clear and it was not until attempts were made to edit it that issues with it became apparent. It appears implementing it in modern non-linear software could be difficult. After much discussion, it was agreed to move the detail to an informative annex and allow alternatives. The authors of this paper generally favour the much simpler approach of using design material properties in the model. The issues are discussed in a paper by Jackson[5].

It could be argued there is a degree of inconsistency in this issue being covered in 1992-2 rather than EN1992-1-1 since it is not specific to bridges. The same applies to several other issues including creep and membrane and shells. It is one of several issues covered in EN 1992-2 because it arises in bridges and is not covered (or in this case was not considered adequately covered) in Part 1. However, many of these issues do sometimes arise in structures other than bridges. Over time, therefore, it seems likely that some material in Part 2 will migrate to Part 1. This has already arisen between ENV and EN stage for fatigue in concrete structures whilst original separate sections in EN 1992-2 and EN 1993-2 on cable stays have merged and moved to a section of EN 1993-1.

In the particular case of the non-linear safety factor format a similar debate arose for EN 1992-1-1 and the approach currently given in EN 1992-2 **Annex PP** was proposed but rejected. Rather than allowing national determination, the section in EN 1992-1 basically leaves it up to users with only very general advice being provided. The UK NA to EN 1992-2 adopts a similar approach (albeit with some more specific warning) which results in Annex PP not being applicable.

Conclusions and Comments

Key decisions of the UK National Annex have been discussed. Only in a minority of cases does the UK depart from recommended values. Few, if any, are really due to genuine physical differences between nations. However, the UK view is that they are all either improvements or indicators of areas where more work is required. As well as deciding UK practice for the next few years, the NA will inform (and in some cases already has informed) UK work on the direction of future improvements on the code. It is hoped that this means the requirements for (and length of!) National Annexes will reduce as the Eurocodes develop.

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EN 1992-2: PD 6687-2: RECOMMENDATIONS FOR THE DESIGN OF CONCRETE BRIDGES

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Abstract

The objective of this paper is to give the background to the development of the provisions of *PD 6687-2: Recommendations for the design of concrete bridges*. The document was prepared with two primary objectives in mind:

(i) Provision of information on topics not covered by EN 1992.
(ii) Provision of guidance where it was considered further explanation of the Eurocode provisions was desirable for their correct and consistent application.

Introduction

The objective of this paper is to give the background to the development of the provisions of *PD 6687-2: Recommendations for the design of concrete bridges*. The document was prepared with two primary objectives in mind:

(i) Provision of information on topics not covered by EN 1992.
(ii) Provision of guidance where it was considered further explanation of the Eurocode provisions was desirable for their correct and consistent application.

The first item was the subject of debate because the principal-based approach used in the Eurocodes together with the wide range of analysis techniques permitted combine to ensure that it is usually possible to design all elements of a bridge utilising Eurocode methodology. It was considered to be undesirable to *require* an increase in the level of complexity of analysis over and above that used in previous practice, although the flexibility to *permit* such analysis was considered to be beneficial. The material included under (i) is therefore usually in the form of design rules that can be applied by hand methods of calculation with a similar level of complexity as required by previous practice to BS5400.

The remainder of this paper looks at each PD entry in turn and provides explanation for the particular requirements given where they are not self-explanatory. References to clauses in EN 1992-2 and EN 1992-1-1 have been abbreviated below. For example, 2-2/3.1(1) is a reference to clause 3.1(1) of EN 1992-2. It should be noted that the clause numbering in the PD does not follow that in EN 1992-2.

PD 6687-2:2008 Recommendations for the Design of Structures to BS EN 1992-2:2005

Actions and Environmental Influences (cl 3.1)

BS 5400 Part 4[1] permitted the effects of imposed deformations (such as settlement and thermal effects) to be ignored at ULS without a check on rotation capacity. BS 5400 Part 4 however required the calculated bending resistance of over-reinforced sections to be reduced by 15% to guard against failure where there might be inadequate rotation capacity to shed these effects. Eurocode 2 differs and requires an explicit check of rotation capacity in all situations, but there is no similar reduction in bending resistance required for over-reinforced sections. In reality, rotation capacity is required even without designed moment redistribution to allow for the differences between calculated moment distribution and real moment distribution arising from differences in modelled and real stiffnesses. To allow for this, a simple rule has been postulated in the PD that recommends only half the plastic rotation capacity from 2-1-1/5.6.3(4) should be utilised in checking whether imposed deformations can safely be ignored; the remainder allows for unintended differences between calculated and real moments.

Partial Factors for Materials (cl 3.2)

Annex A of BS EN 1992-1-1:2004 permits the reduction of the partial material factors in the design of a structure where the tighter limits on tolerances it identifies are met. These tolerances are likely to be tighter than are readily achievable on most construction sites. For new design, the PD highlights that this reduction should only be performed when agreed with the National Authority and when the requirements are clearly identified in the Execution Specification and a quality control system is in place which can guarantee compliance with the stricter limits.

Design Compressive and Tensile Strengths (4.2)

α_{cc} is a nationally determined parameter which is recommended to be 0.85 for bridges but this value was based on calibration against bending and axial force tests only. For shear, it is found that better calibration is achieved with a value of 1.0. However there are a plethora of other situations where it is not clear whether the behaviour is closest to bending or shear, such as in the strut and tie rules. As a result, the NA lists out its value in all situations to try to give the greatest consistency between rules. The PD explains this with examples.

Confined Concrete (cl 4.3)

The rules given in BS EN 1992-1-1:2004 clause 3.1.9(2) allow enhancement of the characteristic compressive strength and ultimate strain limits when triaxial compressive stresses are present due to confinement of the compression area. Such confinement can be provided by link reinforcement or prestressing but no guidance on suitable detailing is given in BS EN 1992-2. It was not intended that this rule be invoked for general calculations on bending and axial force when only the basic detailing rules of EN 1992 have been observed; this clarification was the basis for the PD clause. It was primarily intended for use with concentrated local forces, and loosely forms the basis of the increased resistance in partially loaded areas, where the confinement is provided by the tensile strength of the surrounding concrete. If benefit of confinement by links is to be taken, reference should be made to tests demonstrating that the link geometry proposed can generate the assumed constraint without premature failure of the concrete occurring.

Second Order Effects (6.1)

Unlike in internally post-tensioned bridges, it is possible for externally post-tensioned bridges to buckle between cable deviators under the action of the compressive load. Strictly, the slenderness calculation given in BS EN 1991-1-1:2004 expression (5.13N) should be used to check limiting slenderness (where $\lambda_{lim} = 20 \cdot A \cdot B \cdot C/\sqrt{n}$). This is however an inconvenient process to follow because realistic values of A, B and C are only known after the first iteration of design and their proposed simplified values are often very conservative leading to potentially severe restrictions on maximum deviator spacing. The PD therefore provides a simplification based on previous UK practice found in BD 58/94[2] clause 6.8.8. The BD58 approach uses a slenderness (deviator spacing/section depth) of 12 which is consistent with the slenderness requirements in BS5400-4:1990 for stocky columns. Using a limit of 10 for design to the Eurocodes is therefore slightly more conservative than previous UK practice. Of course, a more accurate value can always be calculated from 2-1-1/(5.13N).

Geometrical Imperfections (6.2)

EN 1992-1-1:2004 and EN 1992-2:2005 provide information on the magnitudes of imperfections but give little guidance on how to apply the imperfections to minimise overall resistance. The PD clause is addressing three main points:

- Generally the disposition of imperfections should be based on the critical mode shapes of buckling (which will typically be taken as the elastic modes)
- It may be necessary to consider several modes to determine the worst effect in a particular element or structure; different elements may have different critical modes
- The modes of buckling considered can usually be idealised by a series of angular deviations. For example, for the pin-ended strut in Figure 1, the sinusoidal buckling mode shape can be approximated as a kink over the half wavelength of buckling, based on two angular deviations, θ_l. This approach is also the basis of the additional guidance given in EN 1992-2 for arched bridges where a deviation $a = \theta_l l / 2$ has to be attributed to the lowest symmetric modes as discussed below. (There is an error in Figure 1 of the PD; *e* should be $\theta_l l/2$.)

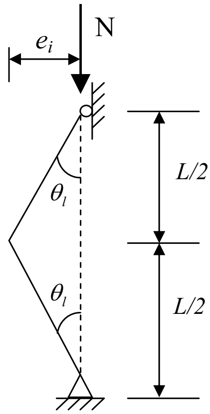

Figure 1. Imperfection for pin-ended strut modelled as an angular deviation

Linear Elastic Analysis With Limited Redistribution (cl 6.4)

Moment redistribution calculation is permitted, indeed effectively encouraged, by Eurocode 2 since ULS rather than SLS will generally govern for flexural design, and the amount of redistribution permitted is closely linked to the rotation capacity of the section in bending. However, shear forces are also altered (albeit usually quite modestly) by moment redistribution and the ability for such redistribution of shear is not explicitly checked in Eurocode 2. As a result, and particularly as shear failure can be brittle, the PD recommends shear to be checked before and after moment redistribution has been taken into account. Such an approach is consistent with past UK practice.

Plastic Analysis (cl 6.5)

EN 1992-2 allows the use of plastic analysis when permitted by National Authorities (see 2-2/5.6.1(101)). The meaning of National Authority in a UK context is explained in the National Annex to EN1992-2 and in clause 2.1 of the PD. As noted in the PD, permission to undertake plastic analysis will generally be established through the processes of Technical Approval used by clients in UK.

Plastic methods of analysis implicitly assume that structures have sufficiently ductility or deformation capacity for a complete collapse mechanism to form before any loss of section resistance occurs in the structure. 2-2/5.6.1(2) effectively expresses this requirement.

Very often, and generally for reinforced concrete structures which are not heavily reinforced or have low-ductility reinforcement, structures will have adequate ductility for plastic methods of analysis to be entirely reasonable. In fact, the lower-bound theorem of plasticity theory underpins other analysis approaches and simplifications explicitly allowed by Eurocode 2. However, whilst EN1992-2 provides provisions that enable the rotation capacity of elements to be determined, as explained in the PD, establishing analytically the demand on ductility for plastic methods to be used is not always straightforward. This is particularly true when upper-bound methods, such as yield-line analysis are used.

The PD goes on to explain some cases where limited rotation capacity might be a concern, such as non-solid slabs and in cases of torsional plastic hinges. The PD also references extensive research by Denton to examine ductility requirements for plastic analysis.

In BS5400-4 the value of γ_{f3} is increased from 1.1 to 1.15 when plastic methods of analysis are used. No equivalent factor is included in EN 1992.

Rotation Capacity (cl 6.6)

The PD provides a brief explanation of the background to the rotation capacity provision of 2-2/5.6.3. It was considered important to highlight two related issues that are not dealt with explicitly in EN1992.

The first is that there is evidence that rotation capacity exhibits a size effect. This possibility is dealt with cautiously in BS5400-4 with a limit on sections depth for which moment redistribution is permitted. No such restriction is provided in the Eurocode, but the PD highlights that it will be advisable to consider rotation capacity requirements carefully for members deeper than the BS5400-4 limit of 1.2m.

The second issue is that the rotation capacity of non-solid sections can be rather less than a solid section of similar size and quantity of reinforcement. This is because an isolated region of concrete in compression tends to crush at a lower compressive strain than the compression zone of a solid section in bending (see also comments on clause 7.1.1 below). Again, BS5400-4 previously dealt with this issue in quite a cautious manner, and the PD explains that, in the absence of more detailed considerations, such a conservative method could still safely be used.

Although the PD highlights cases of potential concern, it was felt important that the PD should not cause undue concern and lead designers to unnecessarily complex analyses. Concrete structures do typically exhibit good ductility and deformation capacity. The PD therefore explains that it will generally be reasonable to assume that the rotation capacity of large or non-solid sections will be sufficient for thermal effects, settlements, and creep and shrinkage to be neglected at the ultimate limit state provided moment redistribution is not used in the analysis.

Analysis of Second Order Effects With Axial Load (cl 6.7)

The effective length provisions given in figure 5.7 of EN 1992-1-1 are somewhat limited in their application to real typical bridge cases. In addition, those cases relating to non-rigid end-conditions require the actual rigidity to be known. This information will not always be available at the time when the compression elements are first being sized. As a result, the effective length provisions from BS5400-4:1990 were imported in this section. They cover real practical boundary conditions and make realistic (and stated) assumptions for end restraint stiffnesses.

Strain Distributions (cl 7.1.1)

A reduced strain limit has to be used either when a section is wholly in compression or when there is a wide flange forming part of a cross section wholly in compression. The need for this in Eurocode 2 stems from the idealisation in Figure 2. The reduction in limiting strain for pure compression arises because the real concrete behaviour is such that the peak stress is reached at a strain approximating to ε_{c2} (or ε_{c3}) and then drops off before the final failure strain is obtained. For pure flexure however, the resistance continues to increase beyond the attainment of this strain because the total force in the compression zone continues to increase. For intermediate cases of strain diagram, the limiting strain needs to be obtained by interpolation between these limiting cases.

The PD considered two aspects of this needed explanation. Firstly, the application of the variable strain limit diagram itself was considered to need some clarification and a figure was provided to do this. Secondly, the definition of a "wide flange" was considered to need clarification to prevent the rule being applied to every small outstand of a cross section which might be wholly in compression. In essence, the variable strain limit should be applied to outstands which make a significant contribution to the overall compressive force. Since trial calculations show that application of this reduced strain limit gives relatively little reduction in bending resistance, a limit of attached flange width of 3 times the flange depth has been provided as a pragmatic limit below which flanges are not wide, which ensures it should not be necessary to consider the reduction for the majority of standard precast beam and slab designs.

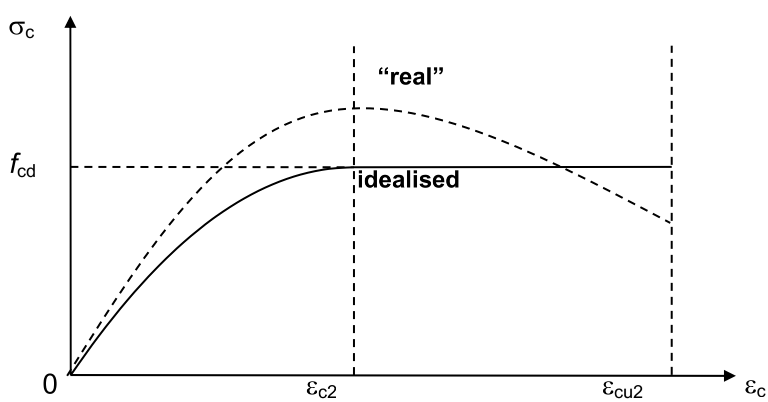

Figure 2. Idealised parabolic-rectangular concrete stress-strain curve and real variation

External Prestressing Strain Between Fixed Points (cl 7.1.2)

This PD clause highlights the differences in requirements between BS EN 1992-2:2005 and BS EN 1991-1-1:2004 for external post-tensioning cables; specifically that deviators cannot be *assumed* to act as fixed points for the cables. The significance of this only really materialises if non-linear analysis is undertaken of an externally post-tensioned bridge to determine its bending resistance. This is because the strain increase in the tendons is dependent on the overall deformation of the bridge between points at which the cable is fixed to it. Shorter distances between fixed points will increase the strain increase of the cable in the tension zones such that as the distance between fixed points tends to zero, the behaviour of the bridge tends towards that for fully bonded prestress. In general, the only points that can be considered as fixed points are the cable end anchorage points.

Robustness of Prestressed Elements (cl 7.1.3)

EN 1992-2 requires that prestressed beams should not fail in a brittle manner due to corrosion or failure of individual tendons. It is desirable for a beam to first exhibit cracking as a warning that there is corrosion occurring. A potential problem arises where tendons are corroding but the concrete remains uncracked. No sign of distress may be apparent if the concrete compensates for the loss of prestress through acting in tension. However, if the concrete suddenly cracks, this tensile strength is permanently lost and the structure may fail suddenly if there is insufficient bending reserve in the remaining tendons and reinforcement.

Brittle fracture can be avoided by ensuring that there is sufficient longitudinal reinforcement provided to compensate for the loss of resistance when the tensile strength of the concrete is lost. This can be achieved by providing a minimum area of reinforcement according to 2-2/(6.101a). This reinforcement is not additional to requirements for other effects and may be used in ultimate bending checks. The disadvantage of this method is that it will always produce a requirement for some reinforcement, even if it ends up being less than that required for other effects. The alternative calculation method (a) (given within the same clause) will usually not give a requirement for any additional reinforcement unless the cross section is unusually lightly prestressed.

Evaluation of Chord Forces for Shear (cl 7.2.1)

Clause 6.2.1 in EN 1992-1-1:2004 allows the designer to take account of the vertical components of the inclined tension and compression chord forces in the shear design of a member with shear reinforcement. These components, V_{ccd} and V_{td}, are added to the shear resistance based on the links. They should, in theory, be determined from the actual chord forces obtained from the truss model and not the value of M_{Ed}/z from beam theory since the latter would overestimate V_{ccd} and underestimate V_{td}, potentially leading to unsafe design in some situations.

Shear Design for Segmental Construction (cl 7.2.4.5)

When joints can open in tension in segmental construction, the PD recommends that h_{red} (Figure 3) be taken as the depth of concrete in compression under the actual applied ultimate loading. This is highlighting the need for considering only co-existent bending and shear effects. If a non co-existent set of effects is considered (ie maximum bending moment with maximum shear force), the depth h_{red} can be significantly smaller than the true depth for co-existing effects and a very conservative design may result.

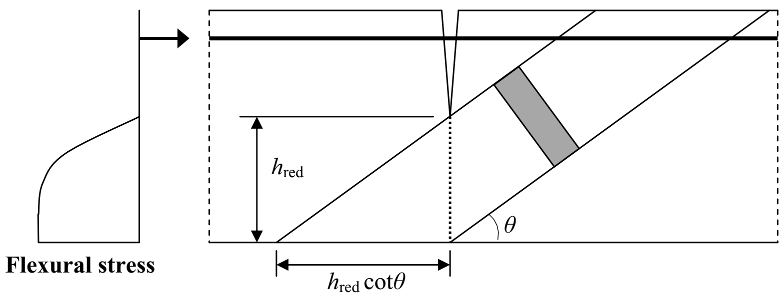

Figure 3. Joint opening in precast segmental construction

Shear Between Web and Flange (cl 7.2.5)

Previous practice for shear flow calculation in BS5400 Part 4 was to use beam theory based on uncracked elastic cross sections. In EN 1992-1-1:2004, the intention is that, for compatibility with the web shear design and the design of additional longitudinal reinforcement for shear, the flange forces and hence shear flow should be determined considering the same truss model. This is a more logical approach and may give rise to larger flange shear flows than from elastic calculation as the latter often implies more of the compression force is in the web and hence that there is less in the flange. The PD therefore provides both confirmation of the need to calculate flange forces from the truss model and clarification of how the flange shear flow can be obtained. More detail on this calculation and the location of the resulting reinforcement is given in reference 3.

Shear at the Interface Between Concrete Cast at Different Times (cl 7.2.6)

For interface shear, a stepped distribution of transverse reinforcement is allowed by BS EN 1992-1-1:2004 but no guidance on the maximum size of these steps is given. The PD recommends that the total resistance within any band of reinforcement should be not less than the total longitudinal shear in the same length and the longitudinal shear stress evaluated at any point should not exceed the resistance evaluated locally by more than 10%. The 10% is the same as that allowed for the provision of shear connectors in EN 1994.

Distribution of Shear Reinforcement for Punching Shear (cl 7.3.2)

Expression 6.52 in EN 1991-1-1:2004 assumes a constant area of shear reinforcement on each perimeter moving away from the loaded area. In bridges, reinforcement is not usually placed like this, but rather on an orthogonal rectangular grid, coinciding with horizontal reinforcement arrangements. Therefore, the area of reinforcement usually increases on successive perimeters away from the loaded area. The PD therefore proposes an alternative approach which allows successive control perimeters to be checked if necessary. In general, shear failure is deemed to occur over a radial distance of 2*d*. Consequently, to enhance resistance, shear reinforcement of area $\sum A_{sw}$ should be placed within an area enclosed between the control perimeter chosen and one 2*d* inside it. To correspond to the 1.5*d* in 2-1-1/(6.52), it is desirable to consider only the reinforcement within a radial band of 1.5*d*. To comply with the need to consider only reinforcement further than 0.3*d* from the loaded perimeter as in 2-1-1/Figure 9.10, only reinforcement further than 0.3*d* from the inner perimeter should be considered. Consequently, only reinforcement further than 0.2*d* inside the control perimeter should be included. These two limits are consistent with the fact that reinforcement at each end of a failure plane is unlikely to be fully effective. This reinforcement zone is shown in Figure 4 of the PD.

If the above method is followed, successive perimeters, u_i, between the basic control perimeter at 2d and the perimeter u_{out} are checked to ensure that the reinforcement in each 2*d* zone above satisfies:

$$\sum A_{sw} = \frac{(v_{Ed} - 0.75 v_{Rd,c}) u_i d}{f_{ywd,ef} \sin\alpha}$$

It will be noted that if the above is applied to the control perimeter at 2*d*, the same total reinforcement requirement as in 2-1-1/(6.52) is produced. If it is applied at the perimeter u_{out}, some reinforcement requirement will still be predicted because of the 0.75 factor on $v_{Rd,c}$ in the expression. This is unfortunate, but as long as reinforcement is detailed so that it is stopped no further than 1.5*d* inside the perimeter u_{out} as required by 2-1-1/6.4.5(4), some reinforcement will be available for this check.

Fatigue in Reinforcement (cl 7.6)

At ENV stage, fatigue was only covered in the bridge part of EC2. However, it was decided that, because fatigue is an issue in other types of structures, coverage should be moved to EN 1992-1-1. One consequence of this was that the service stress range below which fatigue does not have to be checked had to be based on a true non-propagating stress range which is independent of load type. It is possible to derive less conservative rules for specific load types where the number of cycles is limited. The PD introduces rules for highway bridges based on the same work as those used to derive figures in BD 24. However, it was decided considering only the worst span case made them unduly conservative so variation with span is incorporated.

The rules only cover straight bars. However, it seems reasonable to assume the allowable stress range in bent bars reduces proportionately to the stress range given in Table 6.3N of EN 1992-1-1. They also make a distinction according to bar size. This reflects fatigue requirements in BS 4449 and the higher stresses for smaller bars are only allowed for bars complying with BS 4449. This does, however, introduce an anomaly since the full calculations to EN 1992 do not acknowledge an effect of bar size and appear to be conservative for smaller bars to BS 4449.

Checks for specific cases show the rules can be quite conservative compared with the full calculations. Nevertheless the rules are useful as they reduce the number of times full calculations will be required.

Shell Elements (cl 7.7)

It is noted that Annex LL would apply to the design of slabs subjected principally to transverse loading. In previous UK practice, such cases would have been designed using the Wood-Armer equations[4] or the more general capacity field equations[5]. The combined used of 2-2/Annex F, 2-2/6.109 and 2-2/Annex LL to design slab reinforcement does not necessarily lead to conflict with these approaches. The reinforcement produced is usually the same, other than minor differences due to assumptions for lever arms. However, 2-2/6.109 sometimes limits the use of solutions from the Wood-Armer equations or the more general capacity field equations through its limitation of $|\theta - \theta_{el}|$ = 15º. It also requires an additional check of the plastic compression field, which references 4 and 5 do not require. Despite neglect of these requirements, the Wood-Armer equations have been used without difficulty in the past and the PD reflects this experience by permitting their use together with capacity field equations.

Laps (cl 9.1)

There are a number of areas where following the requirements of Eurocode 2 are either impractical or impossible. Transverse reinforcement cannot always be provided at laps in an outer layer for example. The guidance provided in the PD addresses these.

Anchorage Zones of Post-tensioned Members (cl 9.3)

The recommendations given under clause 9.3 have been added since, unlike BS 5400 Part 4, EN 1992 gives few specific requirements for the design of post-tensioned anchorage zones. This is again a function of covering behaviour rather than elements and thus the design must be carried out using the strut and tie rules and rules for partially loaded areas. The reference to the use of CIRIA Guide 1[6] for calculation of reinforcement requirements is legitimate and non-contradictory; although it does not obviously utilise strut and tie analysis, the methods it proposes are based on strut and tie idealisations and the resulting lever arms between tension and compression zones.

Compression Reinforcement of Beams and Columns (cl 10.2)

The rules of EN 1992 were not considered to be completely clear with respect to the detailing of confinement to compression bars designed to contribute to the resistance of the section. The recommendations given in the PD amalgamate the requirements of Eurocode 2 and those in BS 5400 Part 4, which are compatible and more precise.

Pile Caps (cl 10.4)

BS EN 1992-2:2005 does not provide specific guidance for checking shear in pile caps. BS 5400 Part 4 and BS 8110 did. In particular, they provided guidance on the width over which short shear span enhancement could be considered when checking flexural shear across the pile cap (d in Figure 4). This guidance was markedly different; BS 8110 allowing it to be considered over full width provided pile spacing did not exceed 3 diameters and BS 5400 only allowing it to be considered over pile width. This could lead to very large differences between the two codes but there was a lack of test data to resolve this. If you apply the BS 8110 rule to the tests from the paper on which the BS 5400 rules were based[7], punching shear around the loaded "column" becomes critical and it is therefore not possible to tell if the BS 8110 approach is safe. Recent work[8],[9] has addressed this issue and enabled the PD to incorporate guidance which is much closer to that in BS 8110. This work also confirmed that it was not necessary to consider the diagonal plane for corner piles as found in BS5400 Part 4. Figure 4 shows this as plane (c). The provisions should result in significant economy compared with past UK practice.

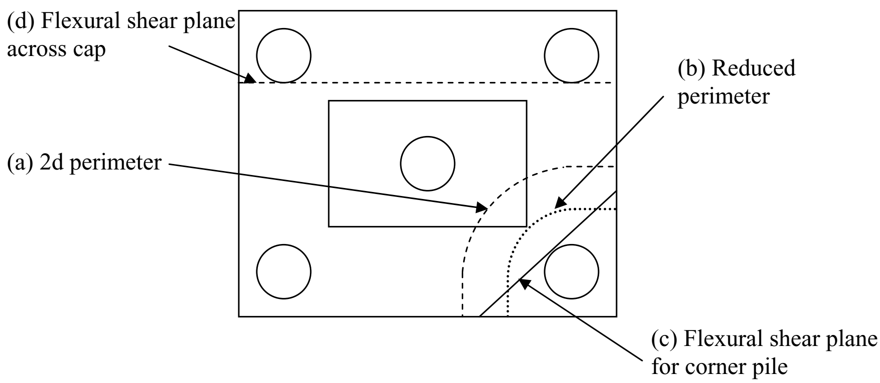

Figure 4. planes to consider for pile cap shear design

Requirements for Voided Slabs (cl 10.5)

Voided slabs are not explicitly covered by EN 1992-2, which is a function of the philosophy of providing rules covering behaviour and not element type. There are sufficient rules in the Eurocodes to cover the design of a voided slab but there was concern that the distortional behaviour of such a slab might not be considered properly by designers due to a lack of direction. As a result, the PD provides additional guidance setting out the recommended analysis to use and the verifications to be performed. The material is mainly imported from BS5400 Part 4. Further guidance on this is provided by work carried out recently by Walker[10].

Additional Rules for External Prestressing (cl 12)

Requirements for replaceability and robustness of external prestressing are not provided in EN 1992-2; the requirements in 2-2/6.1(109) for guarding against brittle fracture need not be applied where tendons are inspectable. In the UK, a more cautious approach has been traditionally adopted to allow for loss of pretress in externally prestressed bridges due to either undetected corrosion or accidental damage. The requirement of BD58/94 to ensure that the bridge can carry dead load with the lesser of two tendons or 25% of the tendons at any section removed has therefore been imported into the PD. In addition, identification of the need to consider the possibility of cable vibration caused by matching natural frequencies of deck and tendons has been introduced.

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Design for Early Age Thermal Cracking

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Abstract

The design of concrete members subject to early age thermal cracking in the UK has previously been carried out to BD28[1] for bridges and BS8007[2] for water retaining structures. These standards have now been replaced by the respective parts of BS EN 1992. CIRIA C660[3] '*Early Age Thermal Crack Control in Concrete*' was published in February 2007 to accommodate changes that had occurred in the UK in relation to the wider range of cements and combinations being used; the development of high performance concretes; and the introduction of the Structural Eurocodes. The use of CIRIA C660 in combination with BS EN 1992 represents a significant step forward in the design methodology for early thermal cracking in the UK.

The principal objective of CIRIA C660 is to support both designers and constructors by providing procedures to limit the extent of cracking to an acceptable level and, where appropriate, to avoid cracking altogether. The report and the associated spreadsheets provide a means for estimating all of the input variables for the design process, including thermal effects and profiles, shrinkage strains, restraint factors and concrete properties, and they set out processes for minimising the risk of cracking and the control of crack widths. These processes are aligned with the Eurocode requirements, extended for use in the UK.

This paper focuses on critical changes in design for early-age thermal cracking in the UK resulting from the introduction of BS EN 1992. These changes relate specifically to the way in which the reinforcement ratio is estimated for calculating the minimum steel ratio and the crack spacing and width.

BS EN 1992 does not fully address design for early-age thermal cracking. CIRIA C660 has been developed to complement the BS EN 1992 design process and provide estimated crack widths that reflect more reliably those observed in practice, so that a robust and serviceable design may be achieved. In the absence of these recommendations, BS EN 1992 could be interpreted to lead to a significant increase in minimum reinforcement, particularly in sections thicker than about 800mm. Conversely, in some other situations the use of BS EN 1992without CIRIA C660 could lead to significantly less reinforcement than required by previous standards BS 8007 or BD28 for controlling crack widths.

The use of CIRIA C660 is therefore recommended in the UK for the design of early thermal cracking effects in combination with BS EN 1992.

Introduction

Early-age thermal cracking (EATC) occurs as the result of restraint to contraction as concrete cools from its peak hydration temperature. In the UK, early-age thermal cracking has previously been dealt with using BD28[1] for bridges and BS 8007 [2] for water retaining structures, supported by CIRIA 91[4] which provided the background to the design method and data for use in the design process. CIRIA 91 was replaced in 2007 by CIRIA C660[3] to take account of the use of a wider range of concreting materials and strength classes; and changes in the design process arising from the introduction of the Structural Eurocodes, in particular BS EN 1992-1-1 and BS EN1992-2 which have replaced BS 8110[5] and BS 5400-4[6] as the general design codes; and BS EN 1992-3 which has replaced BS 8007 for water retaining structures. CIRIA C660 has been cited as Non-Contradictory Complementary Information in the UK National Annex to BS EN 1992-3.

In addition to reflecting changes in the design process (dealt with in this paper) CIRIA C660 also differs from CIRIA 91 as follows;

- Values of temperature change (T_1) for CEM I have been revised and additional information is provided on concretes containing fly ash and ground granulated blast-furnace slag (ggbs)[8].
- Information is provided on autogenous shrinkage which BS EN 1992 now requires '*should be considered specifically when new concrete is cast against old concrete*'.
- Additional information is given on different forms of restraint, how they may be calculated and how they affect crack width.
- Tensile strain capacity is dealt with more comprehensively
- A method for reinforcement design has been developed to deal with cracking caused by temperature differentials in thick sections
- Guidance is given on methods for minimising the risk of cracking
- Advice is provided on specification, testing and in situ monitoring.

In addition, spreadsheet calculators are provided on a CD for estimating temperature rise and temperature differentials; autogenous and drying shrinkage to BS EN 1992-1-1:2004; edge restraint; and reinforcement for crack control.

Review of Previous Design Practice

Prior to the implementation of the Eurocodes in March 2010, design for the effects of early thermal cracking in UK bridges was carried out to BD28.

BD28 firstly required that sufficient minimum reinforcement should be provided so that the force required to first crack the section would not yield the tension reinforcement, as follows:

$$A_s = \left(\frac{f_{ct}^*}{f_y}\right)A_c \tag{1}$$

The area of concrete was taken as the gross cross sectional area unless the section was thicker than 500mm, in which case a surface zone of 250mm at each face is taken.

A further check was then made, to satisfy crack width requirements.

$$A_s = \left(\frac{f_{ct}^*}{f_b}\right) A_c \frac{\phi}{2w}\left[R\left(\varepsilon_{sh} + \varepsilon_{th}\right) - 0.5\varepsilon_{ult}\right] \tag{2}$$

Although given directly as a required area to control cracking, the equation contains elements relating to crack spacing and crack widths, combined into a single formula. By rearranging Equation (2) to give crack width expressed as the product of crack spacing and crack inducing strain as in Equation (3), comparisons with other standards may be more readily made.

$$w = S_{r;\max}\varepsilon_{cr} \tag{3}$$

Where:

$$S_{r;\max} = \left(\frac{f_{ct}^*}{f_b}\right)\frac{A_c}{A_s}\frac{\phi}{2} \equiv 0.5\left(\frac{f_{ct}^*}{f_b}\right)\frac{\phi}{\rho} \tag{4}$$

and

$$\varepsilon_{cr} = \left[R\left(\varepsilon_{sh} + \varepsilon_{th}\right) - 0.5\varepsilon_{ctu}\right] \equiv \varepsilon_r - 0.5\varepsilon_{ctu} \tag{5}$$

For comparison, BS8007 had the following calculation procedure:

$$w_{\max} = S_{\max}\varepsilon \tag{6}$$

Where:

$$S_{r;\max} = \left(\frac{f_{ct}}{f_b}\right)\frac{\phi}{2\rho} \tag{7}$$

and

$$\varepsilon = \left[\left(\varepsilon_{cs} + \varepsilon_{te}\right) - 100\times10^{-6}\right] \text{ or } \\ \varepsilon = R\alpha\left(\Delta T\right) \tag{8}$$

Once expressed in this form it can be seen that the requirements of BS8007 and BD28 were reasonably well aligned, with BS8007 presented in a similar style to the method in the Eurocodes and CIRIA C660. There were, however, some differences in the way in which creep and restraint were accounted for in BD28 and BS8007. Notwithstanding these differences, the principles and the overall approach of BS8007 and BD28 were similar. For completeness, comparisons have been drawn in this paper with both BD28 and BS8007 where possible, relative to Eurocodes.

The Eurocode Design Approach

The Eurocode design method comprises two stages. Firstly, the magnitude of free contraction ε_{free} is estimated and a restraint factor R_{ax} is applied to determine the restrained-strain which, if of sufficient magnitude, may result in cracking. The way in which the restrained-strain is distributed as cracking is then estimated based on the volume and distribution of reinforcement and the nature of the restraint.

BS EN 1992-3 deals with two forms of restraint; continuous edge restraint and end restraint. The nature of the restraint is assumed to influence the way in which cracking develops and different approaches are adopted to estimate the magnitude of crack-inducing strain, i.e. that component of strain which is relieved and exhibited as cracking.

Estimating the Risk of Cracking

Estimating restrained contraction

Similarly to the previous standards, BS EN 1992-3:2006 uses a strain based approach and assumes that all compressive stresses induced during heating are relieved by creep. The restrained contraction ε_r is estimated using the expression;

$$\varepsilon_r = R_{ax}\,\varepsilon_{free} \tag{9}$$

where R_{ax} is the degree of external axial restraint and ε_{free} is the free contraction assuming no restraint. For early-age deformation ε_r is estimated using the expression (10);

$$\varepsilon_r = (\alpha_c .\ T_1 +.\ \varepsilon_{ca})\ K.\ R_{ax} \tag{10}$$

T_1 is the temperature drop; α_c is the coefficient of thermal expansion of concrete; ε_{ca} is autogenous shrinkage; and K is a coefficient for creep. A spreadsheet model for predicting T_1 is provided in CIRIA C660 which derives adiabatic temperature rise curves for a variety of UK concretes. It is based on extensive testing at the University of Dundee [7] and was validated against in situ measurements [3]. Autogenous shrinkage is calculated using the expression of BS EN 1992-1-1.

Estimating the risk of cracking

The risk of cracking is estimated by comparing the restrained strain ε_r with the tensile strain capacity of the concrete ε_{ctu}; for no cracking $\varepsilon_{ctu} > \varepsilon_r$. ε_{ctu} is estimated from the ratio of the mean tensile strength, $f_{ctm}(t)$ and the modulus of elasticity $E_{cm}(t)$ at early-age (the 3-day value is recommended if the specific time of cracking is not known). $f_{ctm}(t)$ and $E_{cm}(t)$ are estimated using the expressions provided in EN 1992-1-1 and coefficients are applied to the ratio $f_{ctm}(t)/E_{cm}(t)$ which take account of both creep (0.65) and the effect of sustained loading (0.8). The net effect of these coefficients is to increase ε_{ctu} under short term loading by 0.8/0.65 = 1.27.

Estimating Minimum Area of Reinforcement

The minimum area of reinforcement $A_{s,min}$ is that which ensures that, if all of the tension in the concrete prior to cracking is assumed to be transferred to the steel immediately after cracking, then the stress in the steel will be below its yield strength. Expressions used by BS 8007 and BS EN 1992-1-1 are shown in Table 1.

In the design approach of BD28 and BS 8007 it was assumed that cracking is initiated from the surface [8]. In practice however, it is more likely that, under conditions of external restraint in which there is tension across the full section, cracking will be initiated at the point where the temperature drop is the greatest, i.e. at the centre of the section (see Figure 1). Stress will therefore be transferred from the full section to the reinforcement when a crack occurs. Hence the underlying assumption regarding the surface zone may not be valid.

BD28	BS 8007	EN 1992-1-1
$A_s = A_c \frac{f_{ct}^*}{f_y}$	$A_s = A_c \frac{f_{ct}}{f_y} = A_c \rho_{crit}$	$A_{s,\,min} = k_c k\ A_{ct} \frac{f_{ct,\,eff}}{f_{ky}} = (k_c k\ A_{ct} \rho_{crit})$
A_c is the gross cross section or a surface zone of 250mm at either edge	A_c is the gross cross section or a surface zone of 250mm at either edge	A_{ct} is the area of concrete in tension
$f_{ct} = 0.12(f_{cu})^{0.7}$	f_{ct} is the tensile strength of the concrete	$f_{ct,eff}$ is the tensile strength of the concrete
f_y is the yield strength of the steel	f_y is the yield strength of the steel	f_{yk} is the yield strength of the steel
		k allows for non-uniform and self-equilibrating stress which leads to a reduction in restraint forces $k = 1$ for h $\leq$ 300mm $k = 0.65$ for h $\geq$ 800mm intermediate values are interpolated
		k_c takes account of the stress distribution in the section = 1 for pure tension

Table 1. Expressions for estimating the minimum area of reinforcement

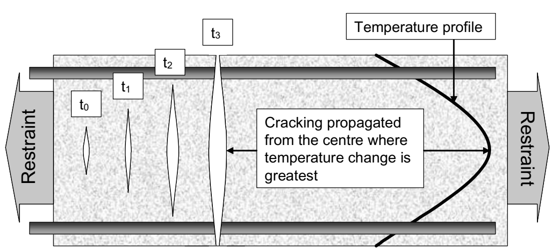


Figure 1. Cross-section through a thick wall subject to external restraint

This revised view of crack development is reflected in the change in EN 1992-1-1 and has been extended for the case of EATC in CIRIA C660 which has increased the coefficient *k* (see Table 1) from 0.65 to 0.75 for h ≥ 800mm to take account of both the (generally) parabolic temperature profile and the fact that in practice some compressive stresses must be relieved by a drop in temperature before tensile stress are generated. A comparison of the effective surface zones is shown in Figure 2.

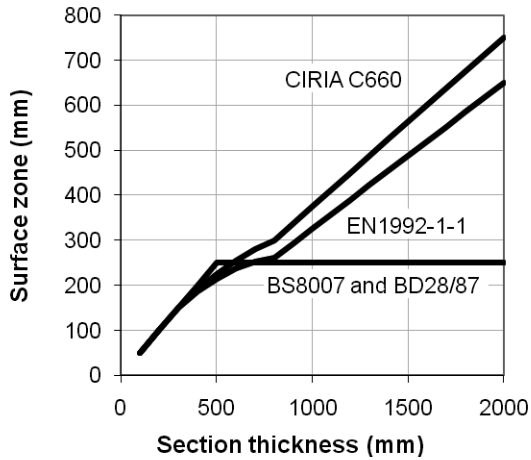


Figure 2. Surface zones used in estimating the minimum area of reinforcement in sections that are dominated by external restraint

The difference is most significant in thick sections that are at risk of cracking due to external restraint, where the methods of EN 1992-1-1 and C660 assume that stress is transferred from a much higher proportion of the section thickness (i.e. a much higher effective surface zone) and hence lead to the requirements for more minimum reinforcement compared with BS 8007 and BD28.

Estimating Crack Spacing and Crack Width

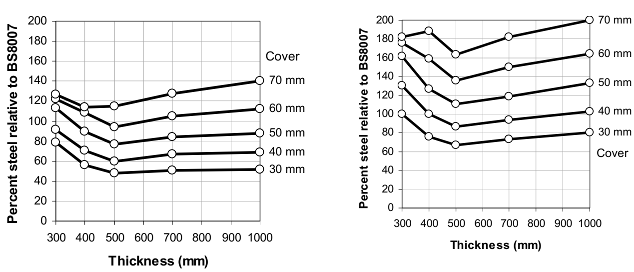
The characteristic crack width w_k (expected to be about 30% higher than the mean value [9], [10]) is estimated from the product of the crack-inducing strain ε_{cr} and the crack spacing $S_{r,max}$.

Crack spacing

The expressions for calculating crack spacing are given in Table 2. The same expressions apply for both edge restraint and end restraint. The second term in the BS EN 1992-1-1:2004 expression appears to be very similar to that of BS 8007. However, the way in which $\rho_{e,eff}$ is calculated leads to very different results. Consider a 500mm thick wall; if c = 40mm and φ = 20mm, the effective surface zone, $h_{e,ef}$ = 2.5(40 +20/2) = 125mm. For a 500mm wall this is only half the value of 250mm used by BD28 and BS 8007. As the value of $\rho_{p,eff}$ is inversely proportional to $h_{e,ef}$ this will result in $\rho_{p,eff}$ being double the value used by BD 28 and BS8007, thus halving the value of the second term in the crack width expression. This difference is partially offset by a cover term (3.4c) but the net effect is that, in this example, the crack spacing estimated using EN 1992-1-1 will be significantly lower than the crack spacing estimated using BS 8007. With no other changes this would lead to a significant reduction in crack control reinforcement compared with BS 8007 as shown in Figure 3 (a).

BS 8007 and BD28/87	EN 1992-1-1
$S_{r,max} = 0.5\dfrac{f_{ct}\varphi}{f_b\rho}$	$S_{r,max} = 3.4c + 0.425\dfrac{k_1\varphi}{\rho_{p,eff}}$
NO cover term	c is the cover (mm)
f_{ct}/f_b is the ratio of the tensile strength of the concrete to the bond strength, which for type 2 deformed bars = 0.67	k_1 is a coefficient which takes account of the bond properties of the reinforcement = 0.8 (and increased in C660 to 1.14)
φ is the bar diameter (mm)	
ρ is the steel ratio based on a surface zone of 250mm or $h/2$, whichever is less	$\rho_{e,eff}$ is the effective steel ratio based on an effective surface zone $h_{e.eff}$ to a depth of $2.5\ (c + \varphi/2)$ or $h/2$, whichever is less
Hence, $S_{r,max} = 0.335\dfrac{\varphi}{\rho}$	and, $S_{r,max} = 3.4c + 0.34\dfrac{\varphi}{\rho_{p,eff}}$

Table 2. Expressions for the calculation of crack spacing



(a) k_1 = 0.8 (EN 1992-1-1) (b) k_1 = 1.14 (CIRIA C660)

Figure 3. The ratio of reinforcement requirements for design to EN 1992 and BS8007 (C30/37 concrete; plywood formwork; limiting early-age crack width to 0.15 mm; cover as shown)

Observations by the authors suggest that the requirements of BD28 and BS 8007, while having been generally applicable, have occasionally led to excessive crack widths and that it would be unsafe to adopt a design that significantly reduces the current requirements. The design parameters were therefore investigated. BS EN 1992-1-1:2004 recommends a factor of 0.7 is applied to bond stress in cases when "good" bond cannot be guaranteed. In C660, this factor has been considered applicable to k_1 in cases of EATC, so the bond coefficient k_1 is increased from 0.8 to 1.14, since 0.8/0.7 = 1.14. Calculations using the increased value of k_1

are shown in Figure 3 (b) and lead to steel requirements that are closer to those of BS 8007 within the normal range of cover. Higher steel ratios than those suggested by BS 8007 are generally associated with high cover.

Crack width

Up to this point in the design the nature of the restraint has not been considered. However, in estimating crack width, BS EN 1992-3:2006 uses different expressions for estimating the magnitude of crack-inducing strain ε_{cr}.

For continuous edge restraint informative Annex M of BS EN 1992-3:2006 assumes $\varepsilon_{cr} = \varepsilon_r$ i.e. the restrained-strain. CIRIA C660 proposes the expression $\varepsilon_{cr} = \varepsilon_r - 0.5\varepsilon_{ctu}$ i.e. ε_r less the residual strain in the concrete after cracking. (This approach is also taken by BD28.) In each case the assumption is that the crack width is strain limited.

For end restraint only a different expression is used as follows;

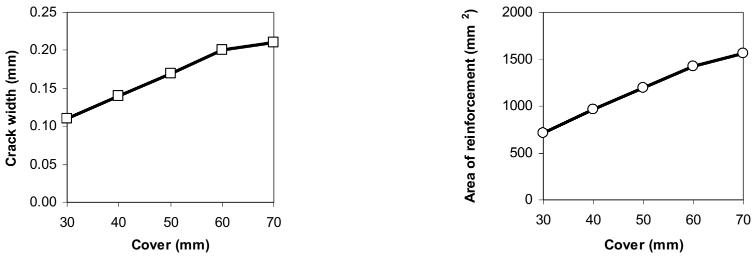
$$\varepsilon_{cr} = \frac{0.5\alpha_e.k_c.k.f_{ct.eff}}{E_s}\left(1 + \frac{1}{\alpha_e\rho}\right)$$

E_s is the modulus of elasticity of the steel; α_e is the modular ratio; $f_{ct,eff}$ is the tensile stress in the concrete immediately prior to cracking; ρ is the steel ratio based on the full area of concrete in tension *[N.B. This is not the same as $\rho_{p,eff}$ used in the calculation of crack spacing]; k* and k_c are area coefficients described in Table 1. This expression assumes that the crack width is limited by the stress transferred to the steel.

Under conditions of end restraint, even when the minimum steel ratio is exceeded, crack widths may be significantly wider than achieved under conditions of edge restraint, although fewer cracks may occur. For example, in a 400mm section with 16mm bars at 250mm centres using C30/37, the crack width resulting from end restraint is estimated to be 1mm, while under conditions of edge restraint the estimated crack width is in the order of 0.15mm.

The Influence of Cover

The net effect on crack width of cover alone is shown in Figure 4. This has been recognised for many years. For example, Campbell-Allen & Hughes [11] recommended that *"the placing of such reinforcement shall be as near to the surface of the concrete as is consistent with the requirements of adequate cover"*. However, in relation to control of EATC, the effect of cover has not previously been quantified. Furthermore, recognising that the crack profile may differ significantly from that of a crack developed by an element in flexure, it may be inappropriate to adopt a similar expression to derive crack spacing and further research is recommended in this area to avoid unnecessarily high volumes of reinforcement being used when high cover is specified.



(a) Effect of cover on crack width

(b) Area of reinforcement (mm^2/m/face) required to achieve a crack width of 0.15mm

Figure 4. The Effect of Cover in a 300mm Wall Subject to a 30°C Temperature Drop and 70% Restraint

Comparison Between Estimated and Observed Crack Widths

Comparisons between observed crack widths and predictions using current methods are shown in Figure 5. These graphs are based on back analysis of measured data, as explained by Bamforth[12]. It is clear that both BS 8007 and BS EN 1992 (without extension as recommended in C660) lead to unsafe predictions of crack width for many of the examples. In some cases the difference was as much as 50%. The method of CIRIA C660, which is based on the method of BS EN 1992 but extended as described, shows a much better correlation with the reported crack widths.

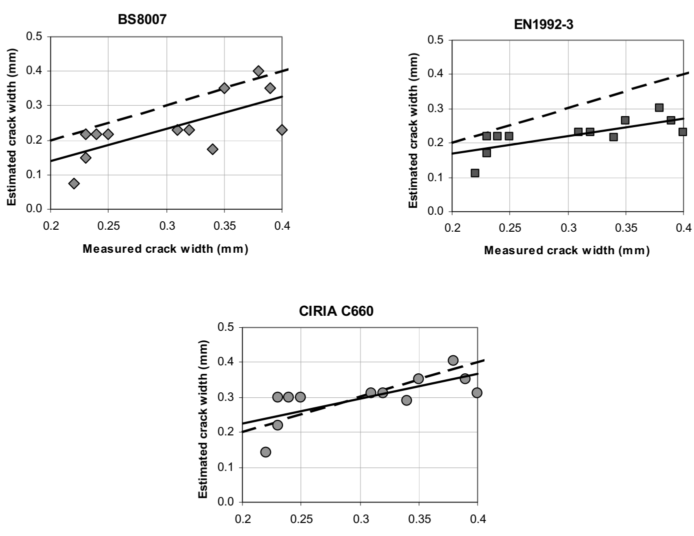


Figure 5. Comparison of observed and predicted crack widths using the methods of BS 8007, BS EN 1992-3:2006 (without extension for EATC) and CIRIA C660[12]

Conclusions

The design approach for early-age thermal cracking adopted by BS EN 1992-3:2006 is broadly similar to that of BD 28 and BS 8007 but there are some significant and important differences as follows;

1) Different values of surface zone are used to estimate the minimum area of reinforcement
2) Different surface zones are used to estimated the steel ratio for calculating crack width
3) BS EN 1992-1-1:2004 includes cover in the expressions for crack spacing and width. This was not included in BS 8007 of BD 28
4) The term f_{ct} / f_b (tensile strength/bond strength) has been replaced by the coefficient k_1
5) Crack development and crack widths depend on whether the element is subject to edge restraint or end restraint and this is reflected in different expressions for calculating crack width
6) Autogenous shrinkage is assumed to occur in all grades of structural concrete

CIRIA C660 has recognised these changes and has proposed modifications to the design parameters to complement EN 1992 and ensure that estimated crack widths reflect more reliably those observed in practice.

Acknowledgements

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DESIGN OF CONCRETE SLAB ELEMENTS IN BIAXIAL BENDING

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Abstract

This paper considers the methods available for designing concrete slabs in biaxial bending at ULS and SLS, with reference to the provisions of EN 1992, and gives guidance on the design of typical bridge slabs.

Notation

M_1, M_2	Bending moment effects per unit width in principal directions
M_x, M_y	Bending moment effects per unit width normal to x and y directions
M_{xy}	Twisting moment effect per unit width in x and y directions
θ	Arbitrary angle of n-axis to x-axis
n	Direction normal to axis of bending considered
t	Direction transverse to axis of bending considered
α_i	Angle of i-th set of reinforcement to the x axis
M_n	Applied moment normal to the n-axis
M_n^*	Bending resistance normal to the n-axis per unit width
$M_{\alpha i}^*$	Bending resistance in the reinforcement direction per unit width
$\varepsilon_{\alpha i}^*$	Reinforcement strain in resistance calculation
ε_n^*	Strain at the level of the reinforcement in the n-direction in the resistance calculation
$\sigma_{s\alpha i}^*$	Reinforcement stress in resistance calculation
$F_{\alpha i}^*$	Reinforcement force per unit width in resistance calculation
F_n^*	Total reinforcement force per unit width in n-direction in resistance calculation
$\left(\frac{A_s}{s}\right)_{\alpha i}$	Area of reinforcement per unit width for i-th set of reinforcement
$M_{1,\max}^*$	Higher principal moment of resistance based on Wood-Armer or similar methods
$\left(\frac{A_s}{s}\right)_{n,eff}$	Equivalent area of reinforcement per unit width in the n-direction
$\varepsilon_{\alpha i}$	Reinforcement strain
ε_n	Strain at the level of the reinforcement in the n-direction
ε_t	Strain at the level of the reinforcement in the t-direction

γ_{nt}	Shear strain at the level of the reinforcement in the *nt*-plane
F_n	Total reinforcement force per unit width in *n*-direction

All other notation is based on the definitions of EN 1992 and its National Annexes.

Introduction

When designing reinforced concrete slabs in bending it is important to remember that slabs generally behave differently from beams. Whilst it might appear convenient to consider the components of bending in the reinforcement directions and verify the ULS bending resistance and SLS criteria only in these directions, such an approach can be incorrect and unsafe. A more considered approach is required.

This paper provides an overview of the issues associated with verifying reinforced concrete slabs in bi-axial bending at ULS and SLS, and discusses the methods available for doing so. Relevant provisions of BS EN 1992 and its National Annex are highlighted.

General requirements for slabs in bi-axial bending

Slabs are generally required to resist principal bending moments (M_1,M_2) that may not be aligned with the reinforcement directions. Consequently the bending effects in a slab are typically expressed as a moment triad comprising a combination of bending moments per unit width and a twisting moment per unit width, expressed in an (x,y) coordinate system as (M_x, M_y, M_{xy}). These bending moments are shown in Figure 1(a) and it is emphasised that here, M_x is used to denote the moment giving rise to stresses in the x-direction. Particular care should be taken when interpreting M_{xy} from the output of analysis software packages, because different sign conventions may be used.

Static equilibrium may be used to calculate the equivalent moment triad in any arbitrary axis system (n,t) where the n axis is at an arbitrary angle θ to the x axis, as defined in Figure 1(b). Equation (1) provides the moment normal to the n-axis, M_n.

$$M_n = M_x \cos^2\theta + M_y \sin^2\theta - 2M_{xy}\sin\theta\cos\theta \qquad (1)$$

If the principal bending moments are both the same sign (e.g. sagging in both directions) then the behaviour is termed <u>synclastic</u> bending. However, the treatment of <u>anticlastic</u> bending, where the principal moments are of opposite sign (one sagging, one hogging), is more complex. In anticlastic bending the twisting effect can strongly influence the cross section analysis, and there will be a combination of compressive strain and perpendicular tensile strain at the same face, which has implications for the verification of ULS and SLS conditions.

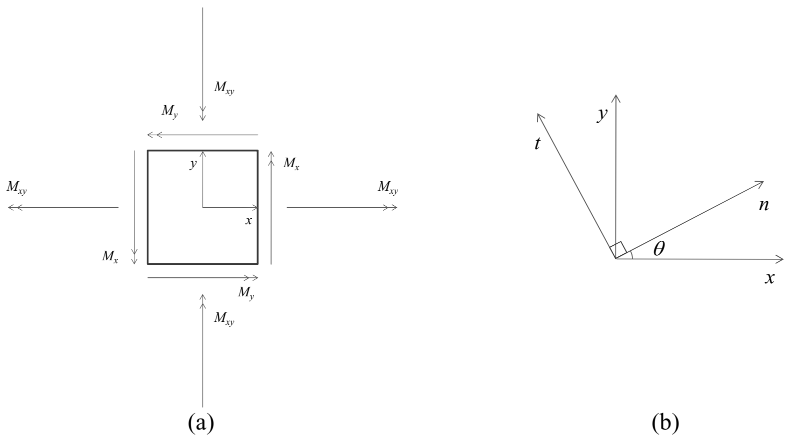


Figure 1. Sign convention and definition of axes

The reinforcement directions in a slab may be either orthogonal or skewed, and do not have to align with the reference axes for the analysis (x,y). Furthermore there are usually (at least) four layers of reinforcement in the slab which can all contribute to the resistance. The angle of the reinforcement to the x axis is denoted as α_i for the i-th set of reinforcement. For example, the typical case with two reinforcement directions at angles α_1 and α_2 is illustrated in Figure 2.

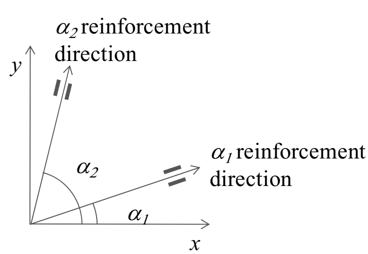


Figure 2. Reinforcement directions

When considering bending at ULS, the objective is to avoid a flexural failure of the slab, which could occur about any axis in the plane of the slab, characterised by the concrete in compression reaching a limiting crushing strain, usually after the tensile reinforcement has yielded.

At SLS it is necessary to verify that the stresses in the reinforcement and the concrete are within design limitations and that the crack widths do not exceed the design limits.

Biaxial bending at ULS

Overview

It has been common UK practice to use the equations presented by Wood and Armer[1,2] for the design of reinforced concrete slabs in biaxial bending. The use of equivalent equations presented by Denton and Burgoyne[3] has now become widespread for assessment.

Importantly, both the Wood-Armer and Denton-Burgoyne approaches recognise the most fundamental difference between the behaviour of beams in uni-axial bending and slabs in bi-axial bending, namely that bending failure of a slab can occur about any axis in the plane in the slab (i.e. in any direction), albeit they do so in a simplified manner.

However, although the use of the Wood-Armer and Denton-Burgoyne approaches will remain valid for many designs to Eurocodes, it has been recognised for some time that they can be unsafe in some cases as they fail to take account of important features of the behaviour of concrete slabs in bi-axial bending. These behaviours are summarised in Table 1.

Behaviour	Features
1. Reduced effectiveness of slabs with skewed, heavy reinforcement	(1a) Neutral axis depth is influenced by forces in all sets of reinforcement; • considering each reinforcement set independently will underestimate the neutral axis depth and overestimate the resistance
	(1b) Bending in certain directions could result in “over-reinforced” failure mechanisms without yielding of the reinforcement; • considering each reinforcement set independently could overestimate the resistance and ductility
2. Reduced effectiveness of slabs in anticlastic bending	(2a) The resistance of the slab is sensitive to the magnitude and form of the applied bending moment field; • neglecting these effects will overestimate the resistance and ductility
	(2b) Concrete strength in compression is reduced due to transverse tension (tension softening) • neglecting this effect could overestimate the resistance and ductility
	(2c) In addition, there is some evidence to suggest that the amount of tension softening may be influenced by a rotation of the compressive stress direction between elastic behaviour and the ultimate limit state.

Table 1. Slab behaviours not predicted by Wood-Armer or Denton-Burgoyne

The significance of these features and approaches to take them into account are discussed in the following sections.

BS EN1992-1-1, **6.1** provides a series of assumptions to use in determining the bending resistance of members in flexure where plane sections remain approximately plane. These assumptions can provide a starting point for considering the bi-axial bending of slabs, but do not address many of the issues highlighted in Table 1.

Methods based on the 'normal moment yield criterion' and 'plane sections remain plane' assumption

Wood-Armer and Denton-Burgoyne methods

The Wood-Armer equations[1,2] were developed to satisfy the normal moment yield criterion, which requires that the moment M_n normal to the n-axis (see Equation (1)) must be less than the bending resistance M_n^* normal to the n-axis for all values of θ, as in Equation (2).

$$M_n^* > M_n \tag{2}$$

Simply stated, the normal moment yield criterion requires that the bending resistance about all axes in the plane of the slab exceeds the applied bending moment.

The Wood-Armer equations also optimise the total area of reinforcement required in the slab in all four layers, and provide the required bending resistances in the reinforcement directions to satisfy these criteria from which the required area of reinforcement can be determined.

An analogous treatment of biaxial bending is provided by Denton and Burgoyne[3], which also uses the normal moment yield criterion but does not require the reinforcement quantities to be optimised, providing more flexibility in design and assessment. Expressions are provided to enable slabs to be verified numerically through determining a factor of safety on the load effects for a given reinforcement arrangement.

Denton and Burgoyne illustrate the problem with plots of moment against the angle θ. The slab is considered to have adequate resistance if the 'capacity' exceeds the 'load effect' for all θ. An example is shown in Figure 3, where despite appearing to be adequate in the x and y directions, the slab would fail in bending under load case 2.

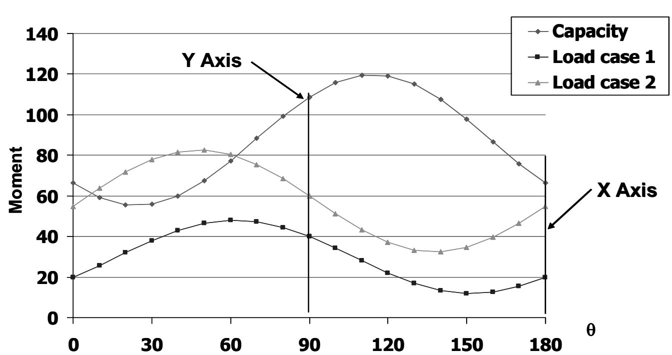


Figure 3. Plots of M_n and M_n^*

To satisfy Equation (2), it is necessary to calculate the normal bending resistance for every angle θ. Wood-Armer and Denton-Burgoyne do this using Johansen's stepped approach[4], based on the bending resistance in the reinforcement directions. For reinforcement aligned with the x and y axes (i.e. $\alpha_1 = 0°$, $\alpha_2 = 90°$ in Figure 2), the normal bending resistance based on Johansen's criterion is as in Equation (3), (noting that in this case $M_{xy}^* = 0$).

$$M_n^* = M_x^* \cos^2 \theta + M_y^* \sin^2 \theta \tag{3}$$

However for the more general case of reinforcement that may be skewed as in Figure 2, the analogous calculation may be carried out using Equation (4)

$$M_n^* = \sum_i M_{\alpha i}^* \cos^2 (\alpha_i - \theta) \tag{4}$$

The use of Equation (4) is based on the assumption that each set of reinforcement is acting independently and that the neutral axis depth in each reinforcement direction is not affected by the presence of the other reinforcement. For normal or light reinforcement the error is small, but for highly skewed heavy reinforcement, the use of this approach may be significantly unsafe as it can overestimate the lever arm or fail to identify over-reinforced situations (*i.e.* it does not address feature (1a) and (1b) described in Table 1).

Simple numerical method to account for interaction of reinforcement and over-reinforced behaviour

In cases of highly skewed and heavily reinforced slabs, it will be appropriate to carry out a numerical calculation of the normal section resistance in each direction rather than using Johansen's stepped approach.

The procedure for carrying out such a calculation can be based on the general principles of BS EN 1992-1-1 **6.1,** but taking account of the strains in the reinforcement directions and the normal components of the reinforcement forces for all sets of reinforcement. Doing so takes account of features (1a) and (1b) of the bending behaviour of slabs described in Table 1.

The reinforcement strain can be related to the normal strain (*i.e.* the strain in the direction perpendicular to the axis of bending) according to Equation (5).

$$\varepsilon_{\alpha i}^* = \varepsilon_n^* \cos^2 (\alpha_i - \theta) \tag{5}$$

Note that because the normal moment yield criterion is being used, the resistance is assumed to be insensitive to the applied moment field, and so there are no components due to transverse bending and twisting included in Equation (5). This assumption simplifies the approach but can lead to some error in the following cases:

(i) when the steel is not yielding, so the reinforcement stress is sensitive to the magnitude of the strain; and,

(ii) when the bending is anticlastic and not aligned with the reinforcement directions, so the top and bottom reinforcement can all be in tension as a result of the applied moment field.

Equation (5) therefore does not take account of feature (2a) in Table 1.
The strain from Equation (5) can then be used to determine the reinforcement stress $\sigma^*_{s\alpha i}$ (checking whether the reinforcement is yielding) and to determine the reinforcement force per unit width, $F^*_{\alpha i}$, for each set of reinforcement, as in Equation (6). The normal component of the reinforcement tension force per unit width can then be calculated and summed for each set of reinforcement according to Equation (7). Once the neutral axis has been determined to give zero net axial force, the bending resistance can be determined based on the lever arm z as in Equation (8).

$$F^*_{\alpha i} = \left(\frac{A_s}{s}\right)_{\alpha i} \sigma^*_{s\alpha i} \tag{6}$$

$$F^*_n = \sum_i F^*_{\alpha i} \cos^2(\alpha_i - \theta) \tag{7}$$

$$M^*_n = F^*_n z \tag{8}$$

Equations (5-7) imply that the reinforcement area may be considered to be effectively reduced by a factor $\cos^2(\alpha_i - \theta)$ if the steel is yielding. However, if the reinforcement is behaving elastically (*i.e.* because it has not reached its yield strain) then the factor effectively becomes $\cos^4(\alpha_i - \theta)$, because of the combined effects of (5) and (7).

A section analysis based on this procedure should be carried out for all values of θ, and the bending resistance M_n^* determined for every direction. This may then be used to verify that there will not be a flexural failure about any axis in the plane of the slab, satisfying Equation (2).

Figure 4 compares the curves for M_n^* using this procedure for different amounts of reinforcement and skew angles. There are two reinforcement directions with $\alpha_1 = 0°$, $\alpha_2 = 90° + \phi$ where ϕ is the skew angle. Equal areas of reinforcement have been considered in both directions. The graphs have been normalised against the higher principal bending resistance obtained from Equation (4), denoted $M^*_{1,\max}$ which is the yield criterion used in the Wood-Armer equations [1,2]and the Denton-Burgoyne equations.

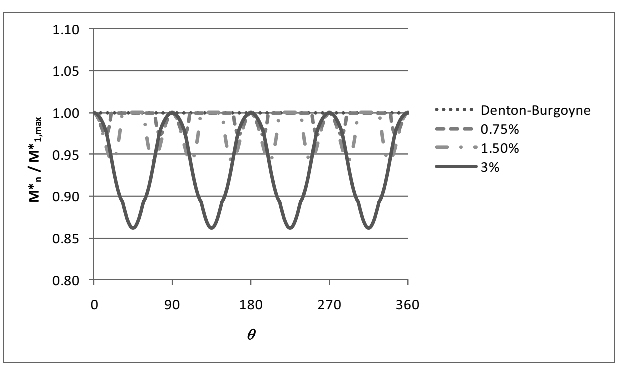
Where the reinforcement proportion is small (0.25%), the results are almost identical to those obtained from the Johansen criterion (as implemented in the Wood-Armer and Denton-Burgoyne treatments) for all skew angles considered. However, for heavier reinforcement there can be a reduction in the peak values of the resistance curves, particularly at high skew angles.

At zero skew it might be expected that the normal resistance would be equal in all directions with equal reinforcement areas, as predicted by Equation (4):
$M_n^* = M_{\alpha 1}^* \left[\cos^2\theta + \sin^2\theta\right] = M_{\alpha 1}^*$. However, as seen in Figure 4(a) there are dips in the resistance in some directions where the method predicts that the reinforcement theoretically would not yield at ULS and the slab is over-reinforced. These reductions are more significant for heavy reinforcement, as expected. For the 3% steel curve the slab is over-reinforced in all directions, including the reinforcement directions, and the worst case is at 45 degrees to the reinforcement.

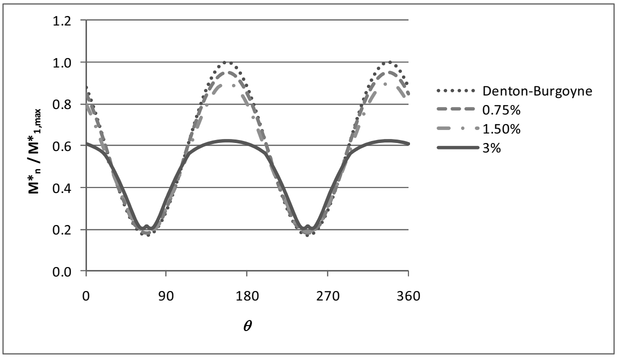
However for the other curves shown with medium amounts of reinforcement the slab is not over-reinforced in the reinforcement directions, or at 45 degrees to the reinforcement directions, but can be over-reinforced in some other directions. These effects would not be identified from section analyses in the reinforcement directions, however in many realistic cases that are not over reinforced in the reinforcement directions the total effect on the factor of safety would be relatively minor.

The major effect seen in Figures 4(b) and (c) is that at high values of skew and heavy reinforcement the peak resistance can be drastically reduced from the value obtained from Wood-Armer, with reductions of over 40% in the peak resistance possible. These reductions are associated with an increase in the neutral axis depth combined with the reinforcement failing to reach yield.

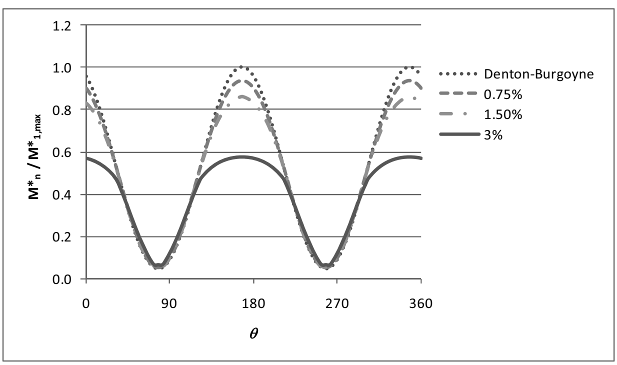
It should be noted that the design is usually not governed by the peak bending resistance, because the curve for the applied bending moment (M_n from Equation (1)) is usually not in phase with the resistance curve (M_n*). Hence the reduction in peak resistance may have an effect on the safety but it is unlikely to be as great a reduction as the reduction in the peak resistance.



(a) Zero skew



(b) 45 degree skew



(c) 65 degree skew

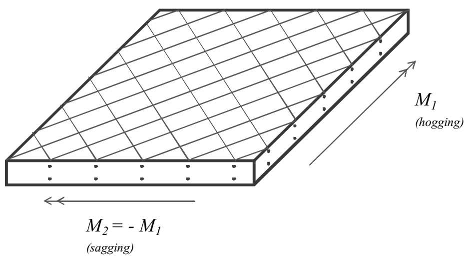
Figure 4. Bending resistance in each normal direction for various skew angles and reinforcement proportions (equal areas of reinforcement in both directions, f_{ck}=45MPa)

It is also worthwhile emphasising that the direct use of the Wood-Armer equations or the Denton-Burgoyne equations based on a transformation of the bending resistance in the reinforcement directions is an appropriate and convenient approximation for many designs, but may be unsafe for highly skewed, heavily reinforced slabs where the use of the stepped yield criterion in their derivation results in a significant degree of non-conservatism.

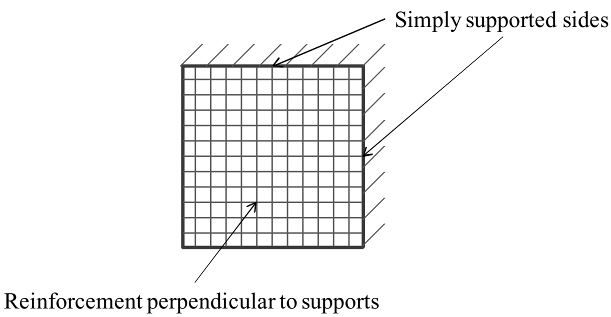
Anticlastic bending

In slabs where there are hogging and sagging moments in orthogonal directions (anticlastic bending), there are some particular complications to be dealt with in determining the resistance. These are summarised in Table 1 for behaviour 2, relating to the sensitivity of the resistance to twisting and transverse bending effects (2a) and the reduction in concrete strength due to tension softening effects (2b) and (2c) (a review of this tension softening effect is found in Vecchio and Collins [5]).

Feature (2a) in Table 1 can be illustrated by considering the extreme case of a pure twisting moment, which is statically equivalent to a principal hogging moment and a principal sagging moment of equal magnitude, applied to a slab with 4 identical layers of reinforcement at 45 degrees to the principal moment directions as in Figure 5(a). This situation might be found in a corner balcony slab, simply supported on two adjacent sides, as in Figure 5(b).



5(a) Principal moments



5(b) Corner balcony slab example

Figure 5. Pure twisting case

By symmetry all four layers of reinforcement must have the same tension in them, and at ULS all four layers will be yielding. As pointed out by Denton[6,7], May and Lodi[8] and others, the depth of compression in the concrete will be about twice what would be predicted by methods based on the normal moment yield criterion, and the resistance can be significantly reduced, particularly for heavily reinforced slabs.

Denton[6,7] discusses some standard cases and shows that in some common cases (where the failure mechanism includes only localised areas of anticlastic bending, e.g. at corners) the total effect of the anticlastic bending on the design is very minor, and standard analysis methods may still be appropriate. However in other cases (e.g. the corner balcony slab in Figure 5(b)) the anticlastic effect can be significant and alternative analysis methods may be necessary. The effect is particularly significant where the principal axes of bending are at a high skew angle to the reinforcement directions (eg 45 degrees).

Sophisticated numerical methods for determining a realistic bending resistance accounting for such effects are presented by Denton[6] and May and Lodi[8]. A mechanism-based approach using wide yield lines that enables the significance of anticlastic bending to be investigated is presented by Denton[7]. These approaches will not be discussed here. As an alternative to methods based on the normal moment yield criterion, EN 1992-2 includes a "sandwich" method for concrete shells in **Annex LL**.

The sandwich method

A slab is essentially a flat concrete shell, which typically has no axial membrane forces. BS EN1992-2, **Annex LL** (informative) provides a sandwich model that can be used for the verification of concrete shell elements.

As illustrated in Figure 6, the sandwich model comprises outer layers which resist the membrane actions arising from bending and any axial forces, while the inner layer resists the out of plane shear forces. Clause **(112)** of **Annex LL** states that the outer layers should be "designed as membrane elements using the design rules of 6.109 and Annex F".

Thus the designer is directed to treat the outer parts of the slab as if they were independent membranes, each one comprising concrete in compression, with a strength that may be determined from Clause **6.109 (103)**, and reinforcement in tension, that may be designed according to **6.109** and **Annex F** (for skewed reinforcement the equivalent equations are given in Hendy and Smith[9]). The thickness of the membrane layers may need to be determined iteratively, and this affects the lever arm and hence the membrane stresses.

However, the National Annex to BS EN 1992-2 slightly relaxes Clause **(112)** of **Annex LL** to allow the use of the design rules of **6.109** and **Annex F**, 'or an alternative realistic membrane element'. This modification avoids some of the complexities that otherwise arise from applying **6.109**, and as PD 6687-**2 7.7** explains, it was made to allow the (more convenient) Wood-Armer equations to be used for the design of slabs in appropriate circumstances.

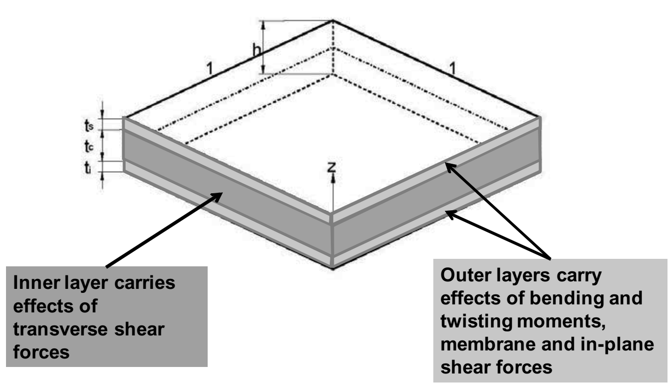
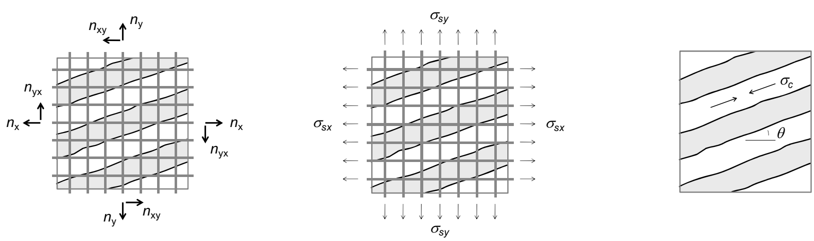


Figure 6. The sandwich model

Using the sandwich model it is possible to treat a slab in bending in a completely different way to the methods described earlier based on the normal moment yield criterion. The use of this sandwich method and the membrane analysis that it requires can be rather complicated, and can at first seem counter-intuitive. For example, for a membrane in tension in both *x* and *y* directions there will still typically also be concrete in compression, which is required to equilibrate any in-plane shear forces in the membrane (see Figure 7). These in-plane shear forces arise in the outer membranes to equilibrate twisting moments applied to the slab.

Note that the sign conventions and notation of **6.109, Annex F** and **Annex LL** are not consistent regarding the direction of the shear, and care must be taken.



(a) (b) (c)

Figure 7. Membrane effects in an outer layer of the sandwich
(a) membrane actions to be resisted
(b) tensile stresses in reinforcement
(c) compressive stress in concrete

For synclastic bending in slabs, this sandwich method will generally predict similar behaviour as the Wood-Armer, Denton-Burgoyne and the simple numerical method described earlier. One of the membranes will be carrying compression in both directions, and will be governed at ULS by the concrete compressive strength, and the other membrane will be carrying tension in both directions, giving rise to yielding of the reinforcement at ULS. The tension membrane will also include some compression in the concrete if there is any twisting in the reinforcement directions, but it is very unlikely that this will govern.

It is the anticlastic case where the sandwich method yields significant differences. In the "pure twisting" example described earlier and illustrated in Figure 4 where the moments are of equal magnitude and opposite sign, the sandwich method correctly identifies the failure mechanism comprising reinforcement yielding in all directions and concrete compression at 45 degrees. In order to calculate the limiting concrete compressive strength in this case, it is appropriate to take account of tension softening behaviour because of the presence of transverse tension (*i.e.* feature (2b) in Table 1).

Account of tension softening can be taken using the approaches and models described by Vecchio and Collins[5], and Denton[6]. Alternatively, and if account is to be taken of the (possible) effect of a rotation in direction of concrete cracks between initial cracking and the ultimate limit state (*i.e.* feature (2c) in Table 1), clause **6.109 (103)** of BS EN 1992-2 should be used.

BS EN 1992-2, **6.109 (103)** is based on work by Carbone, Giordano and Mancini[10]. It covers the reduction in concrete compressive strength due to tension softening in membranes where there is tension in the reinforcement. It also considers the adverse effect of the rotation of the direction of the concrete compression in the membrane from the elastic condition to the ultimate limit state. For the case where reinforcement is yielding, **6.109 (103) Expression (6.112)** gives a maximum compressive stress as in Equation (9):

$$\sigma_{cd\,\max} = \nu f_{cd}\left(1 - 0.032\left|\theta - \theta_{el}\right|\right) \tag{9}$$

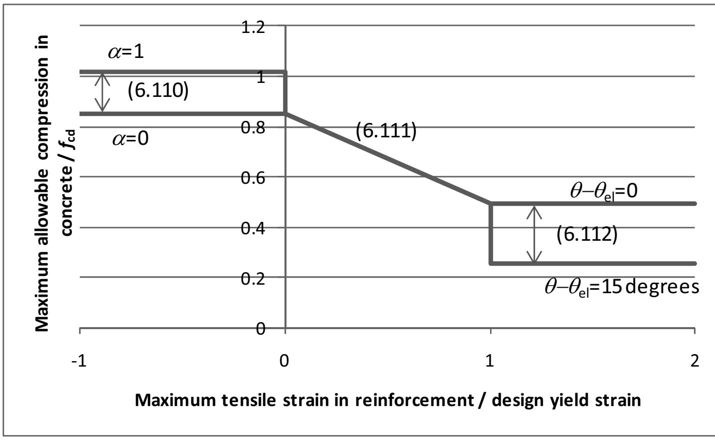
The maximum compressive stress is therefore related to the difference between the elastic principal stress direction and the direction of the concrete stress as assumed in the (plastic) membrane analysis. In the case considered earlier with equal reinforcement and pure twisting both θ and θ_{el} are equal to 45 degrees, so $\sigma_{cd\,\max}$ is obtained from Equation (9) as νf_{cd}. However, in some cases where the difference between the elastic principal stress direction and the final plastic stress direction is at its maximum value of 15 degrees the approach can lead to a severe reduction of the compressive stress by a factor of nearly 2, as given in Equation (10):

$$0.52\nu f_{cd} \le \sigma_{cd\,\max} \le \nu f_{cd} \tag{10}$$

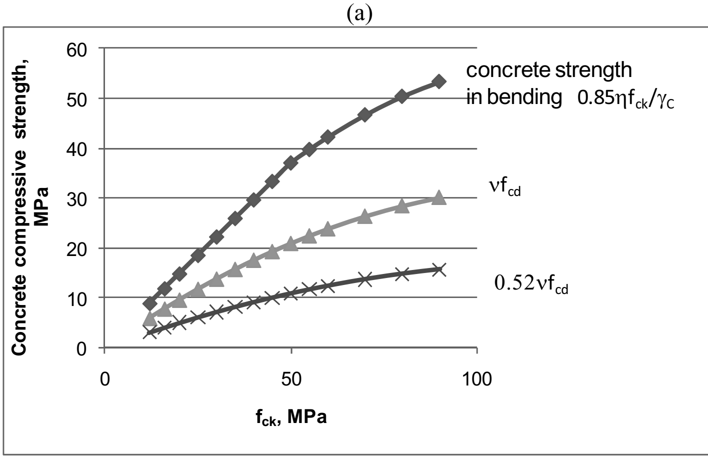
Other limitations apply where there is tension but no yielding (**Expression (6.111)**) and where there is no tension (**Expression 6.110)**, which includes some enhancement of concrete strength depending on the ratio of principle stresses, α. The effect of these expressions is illustrated in Figure 8(a), which shows a maximum variation in the allowable concrete stress of a factor of up to 4, depending on the tension in the reinforcement and the rotation angle, $\left|\theta - \theta_{el}\right|$.

It may be noted that the Expression 6.110 explicitly includes a factor of 0.85, and the UK National Annex therefore gives a value of α_{cc} of 1.0 (see BS EN 1992-2, **3.1.6(101)**) to be used in conjunction with it, rather than the value of α_{cc} of 0.85 used in conjunction with other situations where concrete is in compression arising from bending or axial load. Thus, when α =1.0, the maximum compression in the concrete is consistent with that used in evaluating the bending resistance of a concrete beam.

Figure 8(b) illustrates the range of values from **Expression (6.112)** (see Equation 10) and includes a comparison with the concrete strength normally assumed in the stress block for bending. The reduction in strength can be up to 70%.



(a)



(b)

Figure 8. Concrete compressive stress limits in ULS membrane analyses

Hence the use of the sandwich model for anticlastic bending has two important effects not predicted by traditional bending methods:

(i) the concrete compressive strength is reduced by up to 70%; and,
(ii) tensile reinforcement stresses are included in both faces of the slab simultaneously.

When compared with a traditional bending analysis, there is a greater depth of concrete in compression and a reduced lever arm. The extra steel tension in (ii) also has a significant negative contribution to the bending moment at ULS. The total adverse effect on the calculated resistance can be very significant. The sandwich method provides one possible approach for accounting for this behaviour. Other methods include non-linear numerical models such as those presented by Denton[6,7].

Behaviour	Features	Wood-Armer or Denton-Burgoyne	Simple numerical method in this paper	Sandwich model	Non-linear numerical analysis (eg Denton[6,7]).
1. Reduced effectiveness of slabs with skewed, heavy reinforcement	(1a) Neutral axis depth is influenced by forces in all sets of reinforcement;	No	Yes	Yes (depth of outer layers).	Yes
	(1b) Bending in certain directions could result in "over-reinforced" failure mechanisms without yielding of the reinforcement;	Not fully accounted for	Yes	Yes	Yes
2. Reduced effectiveness of slabs in anticlastic bending	(2a) The resistance of the slab is sensitive to the magnitude and form of the applied bending moment field;	No	No	Yes	Yes
	(2b) Concrete strength in compression is reduced due to transverse tension (tension softening)	No	No	Yes	Yes
	(2c) Tension softening may be influenced by a rotation of the compressive stress direction between elastic behaviour and the ultimate limit state.	No	No	Yes – if 6.109 is used	Only if the effect is included

Table 2. Summary of ULS analysis methods

Since the analysis of anticlastic bending is significantly more complex than standard synclastic bending, it is pragmatic to avoid structural forms with failure mechanisms that are strongly influenced by anticlastic bending (See Denton[6,7]).

ULS Summary

Table 2 summarises the various approaches for analysing biaxial bending of slabs at ULS and describes whether the features listed in Table 1 are modelled. For many typical examples of slab behaviour where the features discussed do not significantly affect the failure mechanism then any the methods listed would be appropriate.

Biaxial bending at SLS

At the serviceability limit state it is necessary to verify the limitations on crack widths and stresses in the reinforcement and concrete, and for slabs in biaxial bending these effects are influenced by the applied moment triad.

EN 1992 does not provide detailed advice on the methods for carrying out these SLS checks for slabs in biaxial bending. The rules for crack width checks in BS EN 1992-2 **7.3.3** and **7.3.4** are only directly applicable in the directions of reinforcement. In a slab the cracking will occur approximately perpendicular to the principal tension direction, which will often be at a skew angle to the reinforcement. While BS EN 1992-1-1 **Expression (7.15)** does provide a basis for the crack spacing in terms of the calculated crack spacings in orthogonal reinforcement directions (see Equation (11) below for reinforcement in *x* and *y* directions) it does not explain how to determine the crack width from this value of crack spacing.

$$s_{r,\max} = \left(\frac{\cos\theta}{s_{r,\max,x}} + \frac{\sin\theta}{s_{r,\max,y}} \right)^{-1} \tag{11}$$

In a beam, crack widths in the direction of reinforcement are calculated using Equation (12). This equation is based on compatibility: the crack width must be equal to the difference between the integral of the tensile strains in the reinforcement and the integral of the tensile strains in the concrete between cracks. These integrals are expressed in Equation (12) as mean strains multiplied by the crack spacing.

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{12}$$

BS EN 1992-1-1 **7.3.4** provides an expression for $(\varepsilon_{sm} - \varepsilon_{cm})$ in the case of beams, accounting for the degree of tension stiffening provided by the bond with the concrete between cracks. The lowest value of $(\varepsilon_{sm} - \varepsilon_{cm})$ allowed (corresponding to maximum tension stiffening) is 0.6 times the steel strain at the crack, whilst the notional upper limit (corresponding to zero tension stiffening) would be 1.0 times the steel strain at the crack.

The approach does not appear to be directly usable in slabs where the crack is at a skew angle to the reinforcement, the principal strain in the concrete does not align with the reinforcement strain, and where there are two sets of reinforcement crossing the crack. It is not clear how much tension stiffening should be allowed in this case.

One approach would be to use a sandwich model and then analyse the outer layers as membranes. However, the procedure for checking membranes at SLS is also a little unclear in BS EN 1992. The note at the end of **Annex F F.1 (104)** appears to provide stress limits which can be used to avoid unacceptable cracking. However a different approach is suggested in **Annex QQ**, which covers control of shear cracking in webs (a directly analogous problem). **Annex QQ** suggests that if there is cracking the reinforcement should be designed according to the normal rules in BS EN 1992-2 **7.3.3** and **7.3.4**, but does not say how this is achieved when the reinforcement is not perpendicular to the crack. Reference may be made to Giordano and Mancini[11] who suggest a method for determining crack widths in membranes, building on work including that by Kaufmann and Marti[12] and Vecchio and Collins[13]. In cases where the design of a slab subject to bi-axial bending is governed by SLS considerations it may well be worthwhile examining these methods.

However, a simpler approach is available, that avoids the need to use a sandwich model. In BS 5400-4[14] the cracking was determined by assuming that reinforcement at an angle $(\alpha_i - \theta)$ to the normal to the crack would have a reduced effectiveness according to $\cos^4(\alpha_i - \theta)$. A similar, but more complete derivation, based on Clark[15] is as follows.

The strain in the reinforcement may be determined from equation (13):

$$\varepsilon_{\alpha i} = \varepsilon_n \cos^2(\alpha_i - \theta) + \varepsilon_t \sin^2(\alpha_i - \theta) - \gamma_{nt} \sin(\alpha_i - \theta)\cos(\alpha_i - \theta) \tag{13}$$

The total tension force per unit width in the *n* direction must then be as in Equation (14), and the equivalent area of steel in the *n* direction is given by (15-16).

$$F_n = E_s \sum_i \left(\frac{A_s}{s}\right)_{\alpha i} \varepsilon_{\alpha i} \cos^2(\alpha_i - \theta) \tag{14}$$

$$\left(\frac{A_s}{s}\right)_{n,eff} = \frac{F_n}{E_s \varepsilon_n} \tag{15}$$

$$\left(\frac{A_s}{s}\right)_{n,eff} = \sum_i \left(\frac{A_s}{s}\right)_{\alpha i} \left[\begin{array}{l} \cos^4(\alpha_i - \theta) \\ + \dfrac{\varepsilon_t}{\varepsilon_n} \sin^2(\alpha_i - \theta)\cos^2(\alpha_i - \theta) \\ - \dfrac{\gamma_{nt}}{\varepsilon_n} \sin(\alpha_i - \theta)\cos^3(\alpha_i - \theta) \end{array}\right] \tag{16}$$

The effective area of reinforcement (and therefore the reinforcement strain) inconveniently depends on the transverse strain and shear strain. However, at SLS, cracking is generally assumed to occur perpendicular to the principal tension, so it is reasonable to assume that $\gamma_{nt} = 0$. The transverse strain is typically unknown at this stage of the calculation, but for synclastic bending where $\dfrac{\varepsilon_t}{\varepsilon_n}$ is positive it is conservative and pragmatic to ignore the effect of this component, and use the simplification in Equation (17), which aligns with the requirements of BS5400:4.

$$\left(\frac{A_s}{s}\right)_{n,eff} = \sum_i \left(\frac{A_s}{s}\right)_{\alpha i} \cos^4(\alpha_i - \theta) \tag{17}$$

For anticlastic bending Equation (17) would not be conservative, because $\frac{\varepsilon_t}{\varepsilon_n}$ has a negative sign. Since the compressive transverse strain will not generally be greater than the tensile normal strain, Clark[11] proposes to assume that $\frac{\varepsilon_t}{\varepsilon_n} = -1$ in this case, thus avoiding the need for an iterative calculation. This approach results in (18).

$$\left(\frac{A_s}{s}\right)_{n,eff} = \sum_i \left(\frac{A_s}{s}\right)_{\alpha i} \left[\cos^4(\alpha_i - \theta) - \sin^2(\alpha_i - \theta)\cos^2(\alpha_i - \theta)\right] \tag{18}$$

Conversely, Hendy and Smith[9] propose that Equation (17) may be used in all cases, which simplifies the procedure. While this could theoretically be unconservative for anticlastic bending cases, it will generally be adequate in practice given that the limit state relates to serviceability and not safety and any beneficial effects of tension stiffening are proposed to be ignored, as discussed later in this paper.

A cracked section analysis can be done based on the equivalent steel area in each of the principal directions to determine the neutral axis depth and therefore the maximum stress in the concrete and the strain at the level of the reinforcement. Once the strain in both principal directions has been determined at the level of the reinforcement, the stress in the reinforcement can be determined from Equation (19).

$$\begin{aligned} \sigma_{\alpha i} &= E_s\left[\varepsilon_n \cos^2(\alpha_i - \theta) + \varepsilon_t \sin^2(\alpha_i - \theta)\right] \\ &\leq f_{yk} \end{aligned} \tag{19}$$

Following this procedure the stress limitations in the concrete and the reinforcement may be verified in accordance with BS EN 1992-1-1, **7.2**, based on the effects of the characteristic combination of actions. The minimum area of reinforcement determined from BS EN 1992-1-1 **7.3.2** may also be compared with the equivalent areas of reinforcement in the crack directions.

For crack width checks a simple and conservative approach would be to assume that there is zero tension stiffening, and that

$$w_k = s_{r,\max}\varepsilon_n \tag{20}$$

where ε_n is the normal strain at the crack at the level of the reinforcement (based on the quasi-permanent combination of actions for reinforced concrete slabs in accordance with the UK National Annex to BS EN 1992-1-1 and BS EN 1992-2) and $s_{r,\max}$ is determined in the *n* direction according to Equation (11).

Conclusions

Biaxial bending should be treated carefully in the design of concrete slabs. It is not generally sufficient to verify bending resistance only in the reinforcement directions without considering twisting effects.

At ULS methods based on the Wood-Armer equations are suitable in many situations, but there are cases where their use can be unsafe, particularly:

(i) where there is heavy, skewed reinforcement
(ii) where the failure mechanism is influenced by anticlastic bending.

Methods are presented to deal with these situations, taking proper account of the depth of concrete in compression, the possibility of over-reinforced behaviour, and the effects of tension softening.

Since the analysis of anticlastic bending is significantly more complex than standard synclastic bending, it is pragmatic to avoid structural forms with failure mechanisms that are strongly influenced by anticlastic bending.

At SLS, BS EN 1992 is not clear on how to carry out the verifications for a slab in biaxial bending. A simple recommended procedure is outlined to verify stress limitations and crack widths.

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SPECIFICATION OF CONCRETE BRIDGES

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Abstract

This paper explains the relationship between the Eurocodes and related product and execution standards. It provides an overview of EN 13670, the execution standard for concrete structures, and discusses the product standards particularly relevant to concrete bridge design and construction. Some background to the implementation of EN 13670 and relevant product standards in UK is also provided, and recent and planned developments described.

Introduction

In developing the rules contained in design standards it is necessary to define requirements for material, products, workmanship and quality control upon which these rules rely. Often these requirements are not included within the design standards themselves. Instead they are included in companion standards. Such an approach has been adopted in the development of the Eurocodes, with these companion standards referred to as product standards and execution standards.

Thus, as indicated in Figure 1, there are essentially three pillars of European standardisation for construction: the Eurocodes are the design standards; the Execution Standards set out requirements that must be achieved during construction; and, Product Standards set out the requirements for material and products. The relationship between these three families of standards is discussed in the European Commission's Guidance Paper L[1].

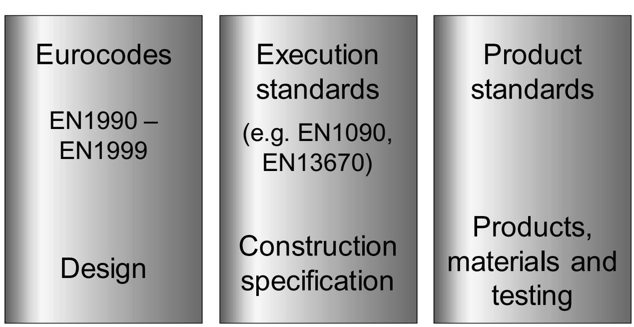


Figure 1. Three pillars of European standardisation for construction

The execution standard for concrete structures is EN 13670[2]. EN 1992[3] assumes that the requirements for execution and workmanship given in EN 13670 are complied with (see EN 1992-1-1, **1.3(1)**). EN 1992 and EN 13670 make extensive reference to product standards, both directly and indirectly. EN 206[4] (concrete), EN 10080[5] (reinforcement) and the (as yet unpublished) EN 10138 (prestressing) are all normative references in EN 1992.

Execution Standard EN13670

Overview

EN 13670 is the Execution standard for all concrete structures. It gives requirements for execution management, falsework and formwork, reinforcement, prestressing and concreting, execution with precast elements, and geometrical tolerances. EN 13670 gives considerable scope for project specific specification. Furthermore, many of the requirements of EN 13670 are quite general, and will not impact significantly on UK practice. There are, however, some implications that are more significant.

The implementation of EN 13670 will require changes to the way in which concrete construction is specified. For buildings, the National Structural Concrete Specification (NSCS) has been updated to align with EN 13670, to provide a set of generalised project specific procedures.

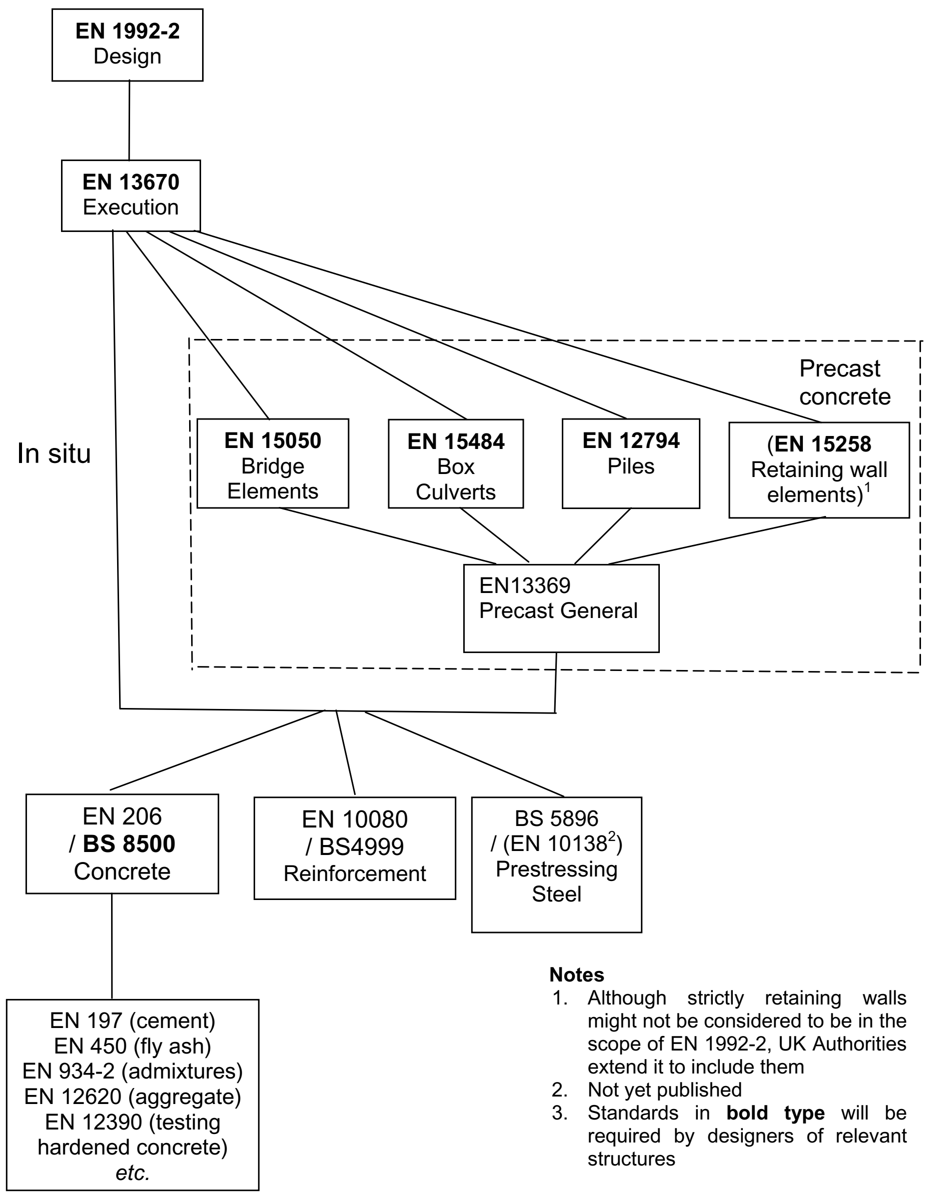
For bridges, although BS 5400[6] included material and workmanship requirements, the majority of structures in the UK have been constructed to the Specification for Highway Works[7] (SHW) Series 1700 or Network Rail Model Clauses[8]. Series 1700 was written as a stand-alone document, in that it is not dependent on BS 5400 Parts 7 and 8. As discussed below, a detailed review has been undertaken to identify required amendments to Series 1700 to align with EN 13670.

Execution management

Execution management is a term introduced in EN 13670. It covers the documentation required for execution, including quality plans and records of compliance, as well as detailing the type and extent of inspection required.

Fundamental to how EN 13670 considers execution management is the selection of an execution class, defined as a classified set of requirements specifying quality levels for the execution of the works (EN 13670, **3.7**).

EN 13670 identifies three execution classes. These are connected with the framework for managing structural reliability established in BS EN 1990:2002, **Annex B**. For each execution class, requirements are given for the inspection of material and products, inspection of execution, and the documentation of inspection.



Notes

1. Although strictly retaining walls might not be considered to be in the scope of EN 1992-2, UK Authorities extend it to include them
2. Not yet published
3. Standards in **bold type** will be required by designers of relevant structures

Figure 2. Relationship between principal standards for concrete bridges

Execution specification

EN 13670 gives various requirements for documentation. In particular, it requires that, before commencement of execution of any part of the works, an execution specification relevant to that part of the works is complete and available (EN13670, **4.2.1(1)**).

The execution specification is defined as all drawings, technical data and requirements necessary for the execution of a particular project (EN 13670, **3.8**). Specific guidance on the information that should be included in the execution specification is given in EN 13670 **Annex A**.

Various parties will contribute to the Execution Specification throughout the project, including the designer and the constructor. Generally it is not expected that the information forming the execution specification would be compiled in a single document or file, but it should be catalogued in a systematic way. The execution specification must be available to those undertaking the work and those involved in supervision and inspection. Generally, apart from very small sites, it should therefore be available on site.

Planned updates to SHW Series 1700

A series of proposed updates to SHW Series 1700 and the related Notes for Guidance (NFG) have been developed to align with EN 13670. The general policy adopted in developing these proposals was to make minimal changes to the structure and content of SHW and NFG.

It is intended that the SHW will form part of the execution specification along with drawings and other design and technical documentation. Some additions are required to Series 1700 to allow this. Moreover certain parts of the present Series 1700 and Series NG 1700 conflict with the general requirements of EN 13670, and require amendment. It is planned that a certain degree of duplication between EN 13670 and SHW and NFG will be retained, but only where it is important for clarity. Where it has been considered desirable, key EN 13670 requirements have been highlighted in Series NG 1700. However, it is not intended that SHW and NFG act as stand alone documents and users will need to be familiar with EN 13670.

The designer will need to ensure that all the required information available at the design stage is included in the execution specification, and as discussed earlier, this information will typically be spread over many drawings and documents. Therefore, to assist the designer, a table will be included in Series NG 1700 which lists the requirements for the execution specification, based on EN 13670 **Annex A**, stating the locations where this information will be found or should be included in the contract documents.

Tolerances

Section 10 of EN 13670 covers tolerances. However, clause **10.1(2)** explains that the tolerances given relate to building structures and that although they may be applied for civil engineering works where relevant, they may be amended in the execution specification. The tolerances included in EN 13670 have been reviewed in detail, and proposals developed for the tolerances to be included in Series 1700.

It is noteworthy that EN 13670 does help address an issue that has been somewhat problematic in UK practice since the introduction of BS 8500[9], in that EN 13670 allows both positive and negative deviation from nominal cover. Positive and negative cover tolerances are included in the proposed Series 1700 updates, with negative deviations towards the concrete surface equal to Δc_{dev} specified in EN1992.

When controlling tolerances during construction, it is important to recognise that apparently acceptable deviations (*i.e.* deviation within specified tolerance limits) can be additive in a structure, and result in overall deviations that are unacceptable. It cannot be assumed, therefore, that satisfying individual tolerance requirement will necessarily mean that all other tolerance requirements will be satisfied; tighter tolerances may well be necessary for specific elements to satisfy all the other tolerance requirements for the structure.

Product Standards

Both BS EN 1992 and EN 13670 make reference to product standards. The key ones for bridges are briefly considered below.

Precast concrete

EN 13670 covers all concrete construction using precast concrete as well as concrete cast in situ. Effectively, as far as EN 13670 is concerned, a precast element is a 'product' just as a reinforcing bar is. Hence, EN 13670 does not cover the *manufacturer* of precast elements although the scope does include the execution of concrete structures which include precast components.

EN 13369[10] is the general standard for precast elements, but there are also more specific product standards for particular types of precast element.

Although the product standards for precast elements are in some respects equivalent to any other product standards, the Construction Products Directive distinguishes between specifications of materials with properties derived from tests and specifications for components with properties determined by calculation. Precast concrete components in bridges (although not always smaller precast components used in other structures) are almost always of the latter type.

The product standard for bridge elements, EN 15050[11], has quite a wide scope so most bridge elements are covered by it. An exception is box culverts which have their own standard EN 14844[12]. Similarly, foundation piles have another standard (EN 12794). EN 15050 adds to rather than replaces EN 13369, so EN13369 is still applicable to the production of precast bridge elements. Like all the precast product standards, it is written with clause numbering consistent with EN 13369.

As can be seen from the basic relationships between the standards shown in Figure 2, although the manufacturer of precast and in situ concrete elements is covered by different standards they both use the same standards for concrete and other constituent materials.

EN 15050 (like other precast product standards) also contains some design information so, unlike most other series of product standards, designers will refer to it quite frequently when designing affected elements. It normally requires the elements to be designed to EN 1992-2.

Early drafts of EN 15050 contained quite a lot of clauses which appeared not to comply with EN 1992-2. Following extensive commenting (including from the UK) this was reduced substantially. However, there are still some areas where EN 15050 allows short cuts (such as for considering continuity in precast beams made continuous for live load) which might not appear to give designs which strictly comply with the normal requirements of EN 1992-2.

These issues are now covered in EN 15050 in informative annexes covering specific forms of construction. Whilst an informative annex in a product standard contradicting EN 1992-2 appears to be an anomaly, from a practical perspective, the Highways Agency has reviewed the EN 15050 material and identified the approaches that it considers can be used without project specific approval. These are intended to be promulgated in the near future.

EN 14844 (Box Culverts) was originally written assuming that design would be undertaken to EN 1992-1-1. However, in many cases, and certainly where the culvert is under a road or railway, UK bridge owners would expect them to be designed to EN 1992-2. The normative reference has now been changed to EN 1992-2. Due perhaps to strong influence of manufacturers, the box culvert standard suggests, in an informative annex, that longitudinal traffic loads due to braking or traction do not have to be explicitly considered. This is an issue that should be addressed in a design rather than product standard, and the EN 14844 guidance does not accord with the requirements of EN 1991-2. The approach is not accepted in the UK, see PD 6694-1[13].

It is not always entirely clear whether precast standards apply to elements which are precast on site rather than in permanent factories. EN 15050 specifically says it applies to ‘precast concrete structural elements produced in a factory’ but does not define a ‘factory’. EN 13369 does the same but adds, ‘this standard may also be applied to products manufactured in temporary plants on site if the production is protected against adverse weather conditions and controlled following clause 6 provisions’.

This implies that, for example, segments of a major glued segmental bridge would normally come within EN 13369 but caissons cast in a dry dock and floated into position would be unlikely to. However, the issue is not as significant as it may appear. For example, EN 15050 does give some specific information on segmental structures even though segments are usually cast on site. This is essentially helpful extra advice rather than (as it might have been) relaxations to allowances for tolerance because of tighter control. It is also in an informative annex. It could therefore be used for segments cast in an on-site facility even though it is not clear this comes within the scope of the document. Similarly, the advice on beam and slab bridges (including significantly on continuity in such structures) is in an informative annex which designers could decide to use (or not) regardless of whether the beams are cast in a ‘factory’ or not. However, where there is ambiguity, the project specification should state whether EN 15050 or EN 13670 applies to the manufacturer of elements which are precast in temporary facilities.

One possible reason the position of the product standards is a bit vague on site precast with EN 13369 appearing to have a wider scope than its product standards is that (unlike EN 13670 and EN 13369) the precast product standards are officially Harmonized Standards and subject to CE marking. It clearly does not make much sense to require CE marking (a system introduced to enable products to be sold across Europe) for elements cast on the same site as they will be used on.

Reinforcement

The EN for reinforcement, EN 10080[5] does not give specific grades. Although the 500 grade now used in the UK is widely used in the more central regions of Europe, there is quite a wide range used, with 600 grade being common in Scandinavia and lower grades used in some southern countries where ductility is required for seismic reasons. Because of this, BS4449[14] has been revised to be consistent with EN 10080 and provide information EN 10080 does not. It only covers the 500 grade usual in the UK.

EN 10080 and BS 4449 give three ductility grades: A (low) to C (high). Some smaller diameter bars and most mesh on sale in the UK is of the lowest A grade; indeed the BS for mesh actually allows a lower ductility then EN 10080. EN 1992-2 recommends that only B or C grade is used for bridges, but leaves it open to national determination in the National Annex. When this was discussed in the BSi work groups and committees, it was notable that everybody who had ever tested concrete elements with low ductility steel was adamant it should not be allowed. It was therefore decided to allow A grade reinforcement in bridges only where it is *not* used in determining ultimate resistance.

There was some resistance to this from precasters who claimed they had difficulty bending B grade links. This seemed hard to explain since higher ductility should make bending easier. After investigation it eventually transpired that the problem was because the different manufacturing process leads to slightly oval steel which tends to go out of plane when bent. This is not directly related to the ductility and it is hoped the manufacturing process can be improved to avoid this in future. Some of the BSi working group and committee members suggested that A grade steel should be banned altogether as the only way to apply the pressure necessary for the development of more ductile mesh.

Prestressing strand

The EN for prestressing strand has been quite controversial and the final EN 10138 has still not been published. It is ironic that this should be one of the last ENs relevant to bridges published since 1860 grade 15.7mm strand to prEN10138 (which was not covered by BS 5896) has been the norm for bridge work since at least 1997. When a draft of EN 10138 received a negative vote, BS 5896[15] was finally amended to cover what has long been the commonest grade used in bridges.

It is not clear if it was intentional but BS 5896 allows a slightly lower 0.1% proof stress than EN 1992 expects. This is significant to design as EN 1992 has a criterion for jacking stress based on 0.1% proof stress as well as one based on ultimate stress and the former tends to govern with BS 5896 steel. This results in a slightly lower limit than was used with BS 5400. Also, EN 1992 (unlike BS 5400) uses the 0.1% proof stress in ultimate strength calculations.

Concrete

The production and specification of concrete itself is covered by EN 206[4] but this is implemented through the complementary standard BS 8500[9] in the UK. BS 8500 is written as a stand alone document and is considered easier to use. It is written in two parts: Part one for the specifier and Part 2 for the concrete producer. It was implemented by the Highways Agency in 2004 so should be familiar to most UK bridge engineers. Originally, both EN 1992-1-1 and BS 8500 gave tables of concrete grades and cover for different exposure classes. This inevitably led to conflict when BS 8500 was amended. To avoid similar difficulties in future, when the NA to EN 1992-1-1 was amended, it was decided to cross-refer to BS 8500 rather than to duplicate the requirements.

BS 8500 also refers to the various ENs and BSs for cement, aggregate, water etc and also for testing but designers will rarely need to refer to these. Recently, and since BS 8500 was last amended, a new section of EN 206 has been published covering self compacting concrete.

Other items

There are also many other ENs and Etags for relevant components. Amongst the more significant are those for prestressing equipment and grouting. When the grouting ENs were first published, which was after the Conrete Society report TR 47[16] was published, they were considered inadequate in the UK. However, they have now been updated[17,18,19] and rely heavily on work done for TR 47.

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[2] BS EN 13670 *Execution of concrete structures.* BSi, London, 2009

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[4] BS EN 206 *Concrete.* BSi, London

[5] BS EN 10080 *Steel for the reinforcement of concrete – weldable reinforcing steel-general.* 2005

[6] BS 5400 *Design of bridges.* BSi, London.

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[8] Network Rail, *Model Clauses*, London.

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[10] EN 13369 *Common rules for precast concrete products* 2006 (inc. corrigenda 2006 & 2007), BSi, London

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[12] EN 14844 *Precast concrete products*: *Box culverts.* BSi, London, 2006

[13] PD 6694-1 *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1: 2004.* BSi, London, 2011

[14] BS4449 *Steel for the reinforcement of concrete – weldable reinforcing steel – bar coil and decoile product – Specification.* 2005 amended 2007 and 2009. BSi, London.

[15] BS 5896 *Specification for High tensile steel wire and strand for the prestressing of concrete* 1980 amended 2007. BSi, London.

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[19] BS EN 447: *Grout for prestressing tendons: Basic requirements*. BSi, London, 2007

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DESIGN ILLUSTRATION – CONCRETE BRIDGE DESIGN

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Abstract

A worked example for an integral precast prestressed beam and in situ slab bridge designed to the Eurocodes has been published as a design guide. The purpose of the guide is to help ease the transition to designing to Eurocodes for UK bridge engineers, in conjunction with the other guidance documents that have been produced. This paper introduces the design example, discusses key issues it raised and makes comparisons with previous UK practice.

Introduction

The Concrete Bridge Development Group decided that an advisory publication[1] was needed to help ease the transition to Eurocode design for UK bridge engineers. The guiding objective of their publication was to help bridge engineers as much as possible, rather than to sell the publication. Because of this, it was designed to complement, rather than compete with, other publications. The only specifically concrete bridge publication known to be in preparation was the Thomas Telford Guide[2]. This is a clause by clause guide. It was felt the most helpful complementary document would be a complete design example. It was further decided that a precast prestressed beam and in situ slab structure would be best to illustrate as many aspects of the code as possible. An integral bridge was used because current thinking favoured such designs.

At the time work was started in 2004, EN 1992-2 itself was not fully finalised, its NA was in very early phases and the decision to issue "PDs" had not been made. It was intended, however, to publish the design example in time for implementation which it was still anticipated would be in time for the intended two year "co-existence" period before withdrawal date in 2010. In the event, implementation got delayed but some of the documents were delayed further. In particular, despite delay in finalising the example, it proved necessary to complete the design example with only a draft of PD 6694-1 *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004*[3]. This meant the coverage of specifically integral bridge issues could not be as complete as originally hoped as it was this part of PD 6694-1 which was most subject to change.

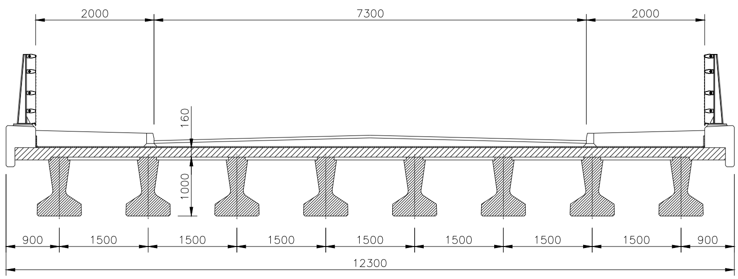
In this paper the design example will be described, some issues it raised discussed and comparison will also be made with BS 5400 design. It is not possible to cover every stage of the design process in this paper. Instead, it will concentrate on those areas which differ most from previous practice or those where significant savings can potentially be made.

Scheme Design

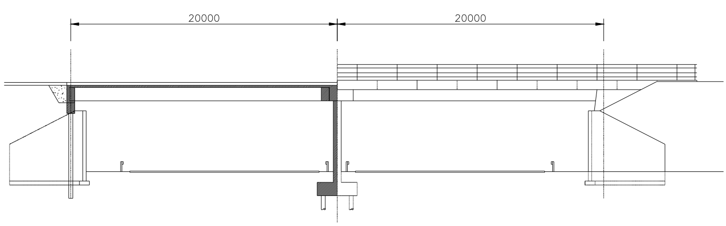
The scheme chosen for the design example was a two span integral bridge, with equal spans each having a length of 20.0m. The bridge carries a 7.3m wide carriageway with a 2.0m wide footway on either side. The superstructure consists of eight standard precast, pretensioned concrete Y beams with a 160 mm deep in-situ reinforced concrete deck slab cast on ribbed permanent GRC formwork. There are in-situ diaphragms at the abutments and pier.

The superstructure is made integral with the substructure. The foundations for the bridge consist of precast concrete piles with in-situ pile-caps. The pile-caps at the abutments are integral with the end diaphragms, while the pier wall is rigidly fixed to both its pile cap and the central diaphragm, avoiding the need for bearings altogether and simplifying the construction.

The integral abutments are small and the piles relatively flexible in order to avoid excessive reactions resulting from thermal expansion of the deck. However, there is still sufficient fill behind the abutment diaphragms to resist longitudinal acceleration and braking forces.



Transverse Section



Elevation (Half in Section)

Figure 1. Details of integral bridge

Model and Analysis

The global analysis of the deck was carried out using a grillage model with eight longitudinal members at 1.5m centres representing the precast beams and associated sections of deck slab, and transverse members at 1.85m centres. The restraint provided by the pier and abutments were represented by rotational springs. The superstructure and sub-structure could have been modelled together in a single 3D model, but the practicalities of the design process mean that they may often be considered separately – in this case, the PD6694-1[3], which was particularly relevant to the substructure design, was not available when the design of the superstructure commenced.

The analysis also had to take into account the construction sequence and the resulting distribution of load. As the bridge does not become continuous until the deck slab and diaphragms are cast and the concrete set, all dead load for the main part of the bridge is carried by the precast beams alone. Therefore, the load effects of the dead weight were analysed using a simple line beam model, pinned at the abutments and pier, and the results added to those of the grillage model. However, this approach was only considered for the serviceability limit state (SLS) – at the ultimate limit state (ULS) the strain discontinuity between the precast and in-situ concrete is not worth considering and all results are obtained from the grillage model.

The flexural stiffness of the concrete members used in the analysis was generally calculated assuming uncracked cross-sections. However, BS EN 15050:2007[4], **Annex D** recommends that cracked section properties be used for the reinforced concrete diaphragm over the pier for the calculation of hogging moments, and this approach was used here.

Materials and Cover

Class C50/60 concrete was used for the precast beams and C35/45 used for all in-situ concrete, with the associated material properties taken from BS EN 1992-1-1:2004[5], **3.1**.

The exposure classes are specified in Section 4 of BS EN 1992-1-1:2004 and BS EN 1992-2:2005[6] and in BS 8500-1:2006[7]. The example bridge is assumed to be passing over a carriageway, and so this is classified as XD3 (exposed to spray containing chlorides). The bridge soffit is more than 5 metres above the carriageway and so according to the UK National Annex to BS EN 1992-2[8], **NA.4.2(106)** does not have to be classified as XD3, though it does not explicitly specify what it should be classified as. XD1 (exposed to airborne chlorides) would appear most suitable and is confirmed by BS 8500-1: 2006. The top of the deck is protected by waterproofing and so the UK National Annex to BS EN 1992-2, **NA.4.2(105)** allows this to be classified as XC3.

The minimum cover for durability is a factor of exposure class and concrete class and type. The UK National Annex to BS EN 1992-1-1[9] specifies that minimum cover requirements should be taken from BS 8500-1:2006, rather than Tables 4.3N, 4.4N and 4.5N of BS EN 1992-1-1:2004. To this minimum cover must be added an allowance for deviation. Values for this were taken from the Highways Agency's Interim Advice Note 95/07 *Revised guidance regarding the use of BS 8500(2006) for the design and construction of structures using concrete*[10], which is slightly more onerous than the minimum values given in the UK National Annex to BS EN 1992-1-1:2004.

The proposed European standard for prestressing steel, EN 10138 *Prestressing Steels*, has yet to receive a positive vote. While it is likely to be published in the future, in the meantime BS 5896:1980[11] has been amended to cover 1860 grade 15.7mm strand.

Reinforcing steel is covered by BS EN 10080:2005[12] and BS 4449:2005[13] – it is the latter that specifies the required properties for the standardised grades.

Actions

Actions can be divided into three main types – permanent, variable and accidental actions.

Permanent actions include self weight, settlement and differential shrinkage. Material densities for calculating self weight can be obtained from BS EN 1991-1-1:2002[14], **Annex A**. Differential settlement was arbitrarily taken as 20mm for this example - A more rigorous estimate should be calculated for a real structure. Generally, the effect of settlement need only be taken into account at SLS, as there is sufficient ductility to justify ignoring it at ULS (BS EN 1992-1-1:2004, **2.3.1.3(3)**). Differential shrinkage, due to the in-situ deck being cast after the precast beams and so shrinking more relative to them, causes tension within the deck slab, compression within the beams and an overall sagging of the deck. As with settlement, provided the bridge has adequate ductility, this effect can be neglected at the ultimate limit state (BS EN 1992-1-1:2004, **2.3.2.2(2)**). Shrinkage is discussed further later in this paper.

Variable actions comprise wind, thermal, construction and traffic loads. Wind loading on heavy, short span bridges such as this is not a critical loadcase, and so was neglected in this example.

BS EN 1991-1-5:2003[15] gives two options subject to National Determination for differential temperature in bridges but, with the options specified in the UK National Annex to BS EN 1991-1-5[16], it is very similar to BS 5400-2:2006[17], having largely been based on this code.

Traffic loads differ more compared to BS 5400-2:2006, but are generally simpler. Before any traffic loads can be applied, the carriageway must be divided up into notional lanes, as specified in BS EN 1991-2:2003[18], **Table 4.1**. For the 7.3m wide carriageway considered in the example, two 3.0m wide notional lanes and a remaining area 1.3m wide are defined. As with BS 5400-2:2006, the positions of these notional lanes do not have to correspond with the lane markings on the bridge. Instead, the lanes and the remaining area are positioned to create the most severe load effect for each element under consideration.

Three traffic load models were applied to the bridge – Load Model 1 (LM1), which is the equivalent to HA loading in BS 5400-2:2006, LM 2, a single axle load, and LM 3, which represents abnormal vehicles.

LM1 comprises a UDL and a double axle loading, referred to as a “tandem system” or “TS” in each lane, and a UDL in the remaining area. Thanks to the nationally determined adjustment factors given in the UK National Annex to BS EN 1991-2[19], this UDL takes a constant value of 5.5kN/m^2 across all lanes and the remaining area, irrespective of the number of lanes or the loaded length. This greatly simplifies analysis.

Load Model 2 (LM2) is a single axle load, 33% heavier than one of the axles of the LM1 tandem system. However, LM2 is not combined with other traffic models so is generally only critical for local effects – for example, when considering transverse flexure or punching shear in the deck slab of this design example.

Load Model 3 (LM3) represents Abnormal Vehicles. A series of load models are defined in the UK National Annex to BS EN 1991-2:2003 for the design of UK road bridges. These will be familiar to those who have used BD 86/07[20]. There are three SV80, SV100 and SV196 representing STGO vehicles with maximum gross weights of 80t, 100t and 196t gross train weight respectively, the last being a 150t vehicle with a separate tractor. There is also a further series of models representing heavier Special Order Vehicles which are equivalent to the old "AIL" vehicles whish were not covered by BS 5400. It should be noted that dynamic amplification factors must be applied to the axle loads of LM3 – these are already incorporated into the other load models. The UK National Annex states that the choice of load model should be determined for each individual project. The choice of SV196 for this design example, which is assumed to be carrying a trunk road, is in line with the best current guidance, the Highway Agency's Interim Advice Note 123/10 *Use of Eurocodes for the design of highway structures*[21].

There is a fourth load model, LM4, which represents crowd loading. By inspection, this load model, which comprises a UDL of 5kN/m^2 across the entire deck, was not critical for this bridge and so was not considered. It appears in the UK it will rarely be critical. It is required in BS EN 1991-2:2003 because with the "recommended values" the LM1 UDL is less severe than LM4 except in Lane 1.

Creep and Shrinkage

The analysis of creep and shrinkage is little changed from BS 5400-4:1990[22] but the derivation of the free strains, particularly for shrinkage, is. BS 5400-4:1990 contained simple figures in the main text for estimating prestress losses and a more detailed appendix which often gave very different results, particularly for timescale. The approach in the BS 5400-4:1990 appendix was based on the 1978 edition of the CEB/FIP model code but that in EN 1992 is based on the 1990 edition[23]. It splits shrinkage into "drying" and "autogenous" components. This gives more realistic results as the two components have very different sensitivities to the various variables considered. The newer approach also uses formulae rather than diagrams which is more convenient when using spreadsheets.

SLS Design

The serviceability limit state (SLS) governs for most prestressed structures. Three sets of checks can be required – decompression, crack widths and stress limits. Reinforced concrete elements only require crack widths and stress limits to be checked. However, SLS considerations will rarely govern for reinforced concrete, particularly as the crack width criteria are less onerous than in BS 5400-4:1990.

In a departure from previous practice, SLS checks even in prestressed concrete, are carried out using cracked section analysis whenever the tensile flexural stress exceeds the effective tensile strength, $f_{ct,eff}$. However, this will normally only arise when either there is no chloride exposure so decompression does not have to be checked or when sections are being checked which do not have tendons close to the tension face. Sections which are checked for decompression and have tendons close to the tension face, such as here, will usually continue to be analysed using an uncracked section. Where cracked sections are considered, it makes the calculations more involved but this is generally not a problem as the cracked section analysis can be done by computer. However, it does mean that the prestress can no longer be calculated directly from a set of equations: as with crack width checks of RC, it is necessary to assume a design and check it

Decompression

For XD (chloride) exposure, the decompression limit is checked under the frequent load combination (i.e. with a reduced "frequent" value of Load Model 1, and no Load Model 3 (abnormal vehicles), as specified in BS EN 1991-2, **Table 4.4b**). All concrete within a distance from the tendons equal to the minimum cover distance required for durability ($c_{min,dur}$) must remain in compression (UK National annex to BS EN 1992-2:2005, **NA.2.2**). For the strand pattern shown in Figure 2, the concrete located $c_{min,dur}$ (30mm) below the lowest tendons remained in compression at midspan. If prestressing tendons had been included near the top of the beam, then decompression of the concrete around these tendons due to hogging over the pier would have had to have been checked as well.

In parallel with the development of the design example, the same bridge was designed to BS 5400 for comparison. It was found that in terms of prestressing requirements, the two designs came out almost identical. However, if the bridge had been passing over a train track or fresh water river, the soffit of the beams would not be exposed to chlorides. Under these circumstances the UK National annex to BS EN 1992-2:2005, **NA.2.2** does not require decompression to be checked under the frequent load combination. Instead, only crack widths need be checked under this load combination, with decompression being checked under the less onerous quasi-permanent load combination (i.e. no traffic load since $\psi_2 = 0$ for traffic actions). It was found that in this case, the level of prestress could be reduced by approximately 25%, or the beam section size reduced, either of which would result in significant savings.

Transfer and tendon arrangement

Along with decompression in service, transfer will often govern the design. BS 5400 had an explicit requirement for concrete that goes into tension at transfer, namely that the tension should not exceed 1N/mm^2. This normally required strand in the top of pretensioned beams. EN 1992 has no equivalent and so the normal rules apply. Without the 1N/mm^2 tension allowance, this can be marginally more restrictive. However, there is an irony that the top strand is there to comply with decompression criterion which only has to be complied with because the top strand is there. If the strand is not provided, it would only be necessary to check crack width at the top of the beam. This would enable you to dispense with the top strand completely, but you would have to provide reinforcement in its place to control crack widths. In purely material terms, this would probably be more economic, if only because the reinforcement could easily be curtailed where it was not needed. However, in the practical situation of precasting, it is not clear this would really represent a saving. For this particular case, it was found that the use of reinforcement in the top of the beam made it possible to eliminate debonding of the lower tendons, which would result in further cost savings. An alternative approach would be to limit the top stress to $f_{ct,eff}$ using strands nearer the top than in Figure 2 but still located low enough down so that the decompression requirement is complied with at transfer.

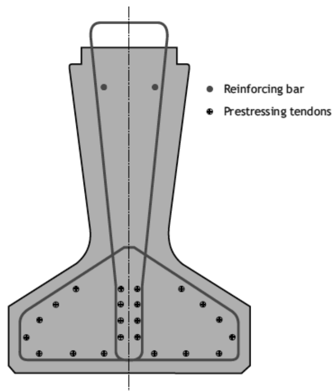


Figure 2. Beam section showing strand positions

BS EN 1992-1-1:2004, **5.10.2.2** places a limit on the maximum concrete stress at transfer of 60% of the characteristic concrete strength at the time of transfer. This may be increased to 70% if "it can be justified by tests or experience that longitudinal cracking is prevented." It is expected that precast concrete manufacturers will use this increase and it was used in this example

BS EN 1992-1-1:2004 explicitly states that the maximum transfer stress is derived from a *characteristic* value of concrete strength – this is a departure from previous practice, where BS 5400 was less specific and a value derived from a small number of cubes (often the lowest of three) was commonly used. It is expected that in practice, rather than casting enough cubes to derive an accurate characteristic strength each time, precasters will use a small number of cubes in conjunction with historic records of the concrete variability that they achieve.

Stress limits

Because the same limit to compressive stress is used in prestressed and RC, the compression limit in prestressed is higher than before. However, this will rarely give any advantage in this type of structure. Cracked section properties will have to be used for prestressed sections where the effective tensile stress is exceeded, but in practice this will cause little extra work for most engineers who routinely use computer programs for such calculations anyway.

Crack widths

For the bridge deck in this example, crack widths only had to be checked in the hogging region of the deck over the central pier (the hogging moment at the abutments being lower) and in the top of the beam at transfer. In principle, crack width in the deck slab in the transverse direction would also have to be considered but, because of the quasi permanent combination considered, this would clearly not be critical. Two methods are presented for checking crack widths. The method involving direct calculation looks somewhat complicated due to the large number of parameters and so this method was used in the design example to demonstrate and, hopefully, clarify the procedure.

There is also a simplified method that avoids direct calculation and it is believed that this method will be used by engineers in most instances. This method tends to be less conservative, apparently because it is based on a lower cover value than is usual on bridges, but for this particular example would have given the same bar spacing as the calculation method. However, ULS and fatigue considerations governed rather than crack widths, as might be expected for the reinforced concrete deck and diaphragms.

ULS Design

Flexure and shear

EN 1992 differs little from previous practice with regards to flexure – unsurprising given that flexure is the best understood aspect of concrete structural design.

There is a significant difference between EN 1992 and BS 5400-4:1990 with regards the calculation of shear resistance. As will have been discussed elsewhere in this conference, the EN 1992 approach will generally result in prestressed members having more shear links than they would have required if designed to BS 5400-4:1990. However, for the design example, this difference was not significant, as it was the requirement for interface shear reinforcement between the precast beams and deck that governed the spacing of the shear links, with B12 bars at 75mm centres required close to the supports, compared to 250mm centres for flexural shear resistance

Fatigue

EN 1992 provides three methods for verifying the fatigue resistance of the reinforcement. The first is to calculate the cumulative fatigue damage due to the predicted cyclic load history of the structure. This is unlikely to be practical for most structures.

BS EN 1992-1-1, **6.8.6(2)** allows a simplified method (which cannot be used for the tendons) whereby provided the stress range in the bars due to the cyclic (live load) portion of the frequent load combination is less than a certain value, k_1, the fatigue resistance can be considered adequate. The recommended value of k_1 = 70 MPa is retained by the UK National Annex, but is increased (albeit in this particular case only to 85MPa) in PD 6687-2:2008[24], **Table 2A** as discussed in the earlier paper on that document. For the design example, the stress in the deck reinforcement over the pier was 128MPa, indicating that the area of steel required for flexural resistance needs to be increased by 50% to provide adequate fatigue resistance.

However, the alternative "damage equivalent stress range" method is significantly less conservative, if more time consuming to carry out. The stress range in the bars or tendons due to a special load model (Fatigue Load Model 3) is calculated. This stress range is then multiplied by a series of factors that allow for site specific factors such as traffic volume or design life. The fatigue resistance is deemed adequate if this modified stress range is less than the permissible stress range for the bar or tendon at N* cycles. For the reinforcement over the pier the permissible stress range is 141 MPa and the calculated damage equivalent stress range is just lower than this, at 140 MPa. It can therefore be seen that significant reductions in reinforcement may be possible through the use of the damage equivalent stress range method compared to the simplified method.

Pier Design

When considering the pier, the forces due to the loading on the bridge deck can be obtained from the support reactions on the grillage model of the deck. In addition, the effects of accidental vehicle impact on the pier must be considered. The loading used is similar in principle to BD 60/04[25] and comprises two components which must be considered together, a main and residual force. Two directions need to be considered – forces parallel to and perpendicular to the carriageway. However, only one direction should be considered at a time. When considering the accidental loads, it must be remembered that action factors are not used and different material partial factors are specified for accidental loads from those used for ULS cases.

Imperfection in the construction of the pier means that axial loads can produce additional moments. BS EN 1992-2:2005 requires that this is taken into account in the analysis in two ways. Firstly, it is assumed that any axial load is applied with an eccentricity of 1/30th of the depth of the section (but not less than 20mm). Secondly, the column is assumed to deviate slightly from the vertical, while the axial force remains vertical. These effects produce additional moments that are additive to those obtained from the grillage model of the deck.

Tall, thin piers may buckle under axial loading. The likelihood of this is related to the slenderness of the member. Provided the slenderness of an axially loaded member is below a limiting value then buckling does not need to be considered – the pier considered in the design example was relatively stocky, with a length to depth ratio of 10.2 and so buckling could be neglected.

Conclusion and Comments

This paper has considered the design of a prestressed beam and slab concrete integral bridge designed to the Eurocodes. This design has been published as a worked example and accompanying commentary to assist UK bridge engineers in the transition to Eurocodes. Some of the key conclusions that can be drawn from the design example are:

i) For structures subject to de-icing salt or marine environments, the design of prestressed sections of highway bridges to BS EN 1992-2:2005 comes out very similar to BS5400. It appears from other calibration work that this will always be the case for “normal” structures where the tendons are reasonably close to the tension face at the critical sections.

ii) Where not exposed to chlorides, significant savings can be made in the amount of prestress required when compared to BS 5400.

iii) The simplified method for carrying out fatigue checks appears to be very conservative, and significant savings in reinforcement quantities will be possible in many situations if the “damage equivalent stress range” method is used instead.

However, the most important conclusion is that the design of concrete bridges to the Eurocodes is no harder than it was to BS 5400, it’s just different.

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SESSION 2-1:

EN 1993 – STEEL

Bridge Design to Eurocodes: UK Implementation
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EN 1993: OVERVIEW OF STEEL BRIDGE DESIGN TO EN 1993

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Abstract

The objective of this paper is to give an overview to the use of EN 1993 for bridge design since there are many more parts which need to be used than there were in BS 5400, where all the rules necessary for steel bridge design were contained in BS 5400 Part 3. This paper identifies the parts of EN 1993 required to design a typical steel bridge (which in the UK is most usually of steel-concrete composite construction and hence reference is also made to EN 1994) and highlights areas of difference to practice in BS 5400.

1 Introduction

There are 58 Eurocode parts. Steel-concrete composite bridge design is the activity requiring the largest fraction of these as indicated in Figure 1. Additionally, the UK National Annexes will be required for each part and BSI Published Documents (PDs) have been made to accompany most of the various Eurocode parts, providing background to the rules and recommendations for their implementation. For steel bridges, the main one of these PDs is PD 6695-2[1] which is the subject of a separate paper. Whilst this might seem daunting to those with limited knowledge of the Eurocodes, pilot studies by consultants have indicated that Eurocodes 2, 3 and 4 are simply different from BS 5400, not more difficult to use. Eurocode 3 looks particularly daunting with its 17 parts, but Eurocode 4 has a much more familiar feel to it while Eurocode 2 is a much more comprehensive document than BS 5400 Part 4[2].

The individual designer will notice some cultural differences when first using the Eurocodes and particularly those parts of EN 1993. Engineers will need to make greater use of first principles, as fewer rules and formulae are given. In many cases, this will lead to greater use of finite element modelling. There is inevitably some new terminology and the term "Eurospeak" has been coined by some to describe this. However, initially perplexing requirements such as *"verify web breathing under the characteristic combination of actions"* will quickly become second nature if the results of pilot studies are anything to go by. The remainder of this paper describes some of the differences between the requirements of the steel Eurocodes and existing UK practice.

2 Steel and Steel-Concrete Composite Design to EN 1993-2 and EN 1994-2

Material properties

Unlike EN 1992, where designers need to get used to using cylinder strength rather than cube strength in calculations, there is little of any great difference to contend with in EN 1993 with respect to material properties. Yield strength varies with plate thickness in the same way as it does when using BS 5400 Part 3[3], and limiting plate thicknesses to prevent brittle fracture come out very similar to those obtained from BS 5400 Part 3, although the calculation route is a little different. Some additional guidance is provided in EN 1993-1-10 as to when to specify steel with improved through thickness properties (Z quality steel to BS EN 10164) in situations where a plate can be loaded through its thickness. These rules have however been universally panned by the UK steel industry; not for being incorrect, but because they tend to encourage specification of expensive steel, rather than consideration of better detailing. This is unusual for the Eurocodes, which usually promote greater thinking in design. As a result, the National Annex to EN 1993-1-10 and its companion document PD 6695-1-10[4] make some substantial, but permissible, modifications to the Eurocode recommendations

Global analysis

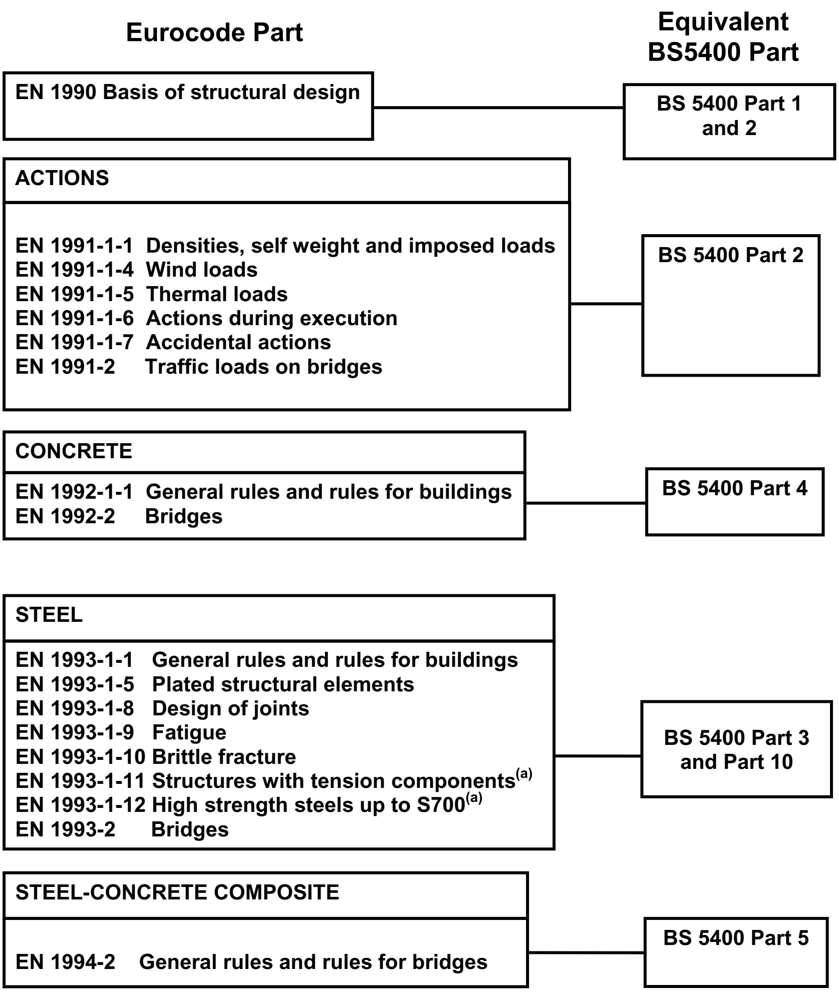
There has been concern expressed about the increase in complexity of global analysis to EN 1993 and EN 1994. It is true that the default analysis in the Eurocodes is second order, considering P-Delta effects. However, the exceptions where second order analysis need not be used and where first order analysis will suffice are such that in almost all situations where first order analysis was used in previous practice to BS5400, it can still be used. The effects of local plate buckling and joint flexibility theoretically need consideration in global analysis, but, once again, the effects have to be so severe before they require inclusion that generally they can be ignored. Similarly, shear lag needs to be considered but frequently will not lead to any actual reduction in acting flange width. For composite bridges, the effects of cracking on stiffnesss in global analysis are treated in almost exactly the same way as in BS5400 Part 5[5].

Section classification

The section classes employed in EN 1993 will look familiar to UK building designers but less so to bridge designers in that there are four section Classes:

- Class 1 cross sections can mobilise a plastic bending resistance and have enough rotation capacity to permit plastic global analysis;
- Class 2 cross sections can mobilise a plastic bending resistance but have insufficient rotation capacity to permit plastic global analysis;
- Class 3 cross sections can achieve a bending resistance corresponding to first yield;
- Class 4 cross sections can achieve a bending resistance corresponding to plate buckling in compression somewhere at a stress below that of yield.

Class 1 and 2 cross sections were previously referred to as “compact” in BS 5400 Part 3. The distinction between them is lost somewhat for bridge design to EN 1993-2 as plastic global analysis is not allowed, other than for design situations involving accidental actions. Class 3 and 4 were previously referred to as “non compact” in BS 5400 Part 3, although a similar distinction within “non-compact” existed but was hidden; the equivalent to a Class 4 cross section was essentially one where a reduction was needed to the web thickness in stress analysis. The boundary between compact and non-compact behaviour remains very similar to that in previous UK practice.



Notes: a) Not covered by BS 5400.

Figure 1. Eurocode parts needed in the course of the design of a steel-concrete composite bridge and the nearest equivalent British Standards

Shear lag in cross section design

One significant conceptual difference between EN 1993 and BS 5400 is that shear lag must be considered at the ultimate limit state. Previous UK practice was to ignore this at ULS on the basis that the occurrence of plasticity led to redistribution of stresses across the cross section. The approach in EN 1993-1-5 is to require consideration of shear lag at ULS, but to make explicit allowance in calculation for this plasticity. Consequently, different effective widths are used for calculations at ULS and SLS. Initially, those keeping an eye on the drafting of EN 1993 in the UK objected to the possible loss of economy implicit in using a reduced flange width at ULS, but trial calculations showed that most typical bridge geometries would achieve a fully effective flange at ULS in any case. The difference between effective widths to be used at SLS and ULS is illustrated in Figure 2 for the fraction of flange width acting at an internal support of a continuous steel beam with span lengths L and flange width b_o available each side of the web. The UK National Annex to EN 1993-1-5 selects the method (from the available methods listed) that minimises the influence of shear lag at the ULS. If a suitable FE model with shell elements is used for the analysis, the reductions need not be made as they will be included in the modelling. Clearly an economic design will only result if non-linear analysis is undertaken to allow for the redistribution mentioned above.

Shear lag must also be considered for concrete flanges of steel-concrete composite beams, but the same effective width is used for SLS and ULS for simplicity. This has little implication for bending resistance as there is usually an excess of available compression concrete. There can however be benefits in using a smaller effective width in shear connection design as it can lead to fewer shear connectors being required.

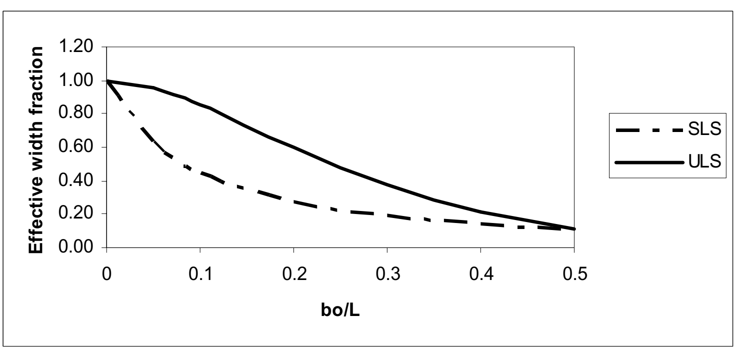
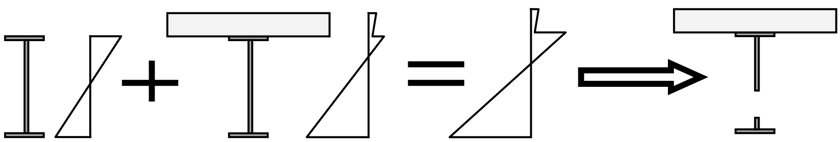


Figure 2. Effective width of un-stiffened steel flange at an internal support

Cross section resistance for bending

For Class 1 to 3 cross sections, the calculation procedure for bending resistance will be found to be very similar to the design of compact and non-compact beams to BS 5400. One small difference for Class 3 cross section design is that where the bending resistance is based on first yield at an extreme fibre, EN 1993 defines an extreme fibre as the mid-thickness of the flange, rather than the outer surface. This gives a small increase in economy over BS 5400, particularly for shallow beams with thick flanges. EN 1994 employs a similar rectangular stress block for concrete for plastic design as is used in BS 5400 Part 5, although the resisting compressive stress is slightly higher. The treatment of Class 4 cross sections and beams with longitudinal stiffeners (which are treated as Class 4 cross sections in EN 1993) differs significantly, however, from that in BS 5400 Part 3.

Class 4 beams without stiffeners are treated by making reductions to the compression areas and then checking stresses against yield when calculated on the resulting reduced cross-section. The procedure for composite beams is to first calculate accumulated stresses on the gross cross section, following the construction sequence, and then to determine the effective areas of the compression elements based on this stress distribution. Finally, the accumulated stresses are recalculated using the reduced effective steel cross section at all stages of construction. This is illustrated in Figure 3.



(i) Accumulated stresses determined on gross cross section

(ii) Accumulated stresses used to determine an effective steel cross section for re-calculation of accumulated stresses

Figure 3. Illustrative procedure for determining effective cross section in Class 4 composite beams

Class 4 beams with longitudinal stiffeners are treated in the same way as beams without longitudinal stiffeners in EN 1993, unlike in BS 5400 Part 3 where a completely different approach to calculation was employed. In BS 5400 Part 3, individual panels and stiffeners are checked for buckling once stresses are determined in them, generally using gross cross sections other than for flange plates as shown in Figure 4. There is therefore limited load shedding between components and a single overstressed component can govern the design of the whole cross section. In EN 1993-1-5, effective widths are again used to allow for buckling of web and flange elements, as for unstiffened Class 4 cross sections, but the same approach is also used for stiffeners. This effectively allows load shedding between all the various elements such that their combined strengths are optimally used. This represents a significant change from previous UK practice and can give rise to an increase in economy in the design of longitudinally stiffened cross sections. A typical resulting effective cross section is shown in Figure 5. Reference 6 gives greater background to these differences.

Shear buckling resistance

The rules for shear buckling in EN 1993 and BS 5400 are based on quite different theories but produce similar results. In the early 1970s, two contemporaries worked on the problem; Rockey in the UK, and Höglund in Sweden. Rockey's theory was adopted in BS 5400 Part 3 but now, thirty years on, Höglund's theory is being used in EN 1993-1-5. The implications are, as previously stated, not great for the design of webs themselves, but are significant in the design of transverse stiffeners as Höglund's theory places less demand on their strength. This is reflected in EN 1993-1-5, which allows lighter transverse shear stiffeners to be designed than would be permitted to BS 5400 Part 3.

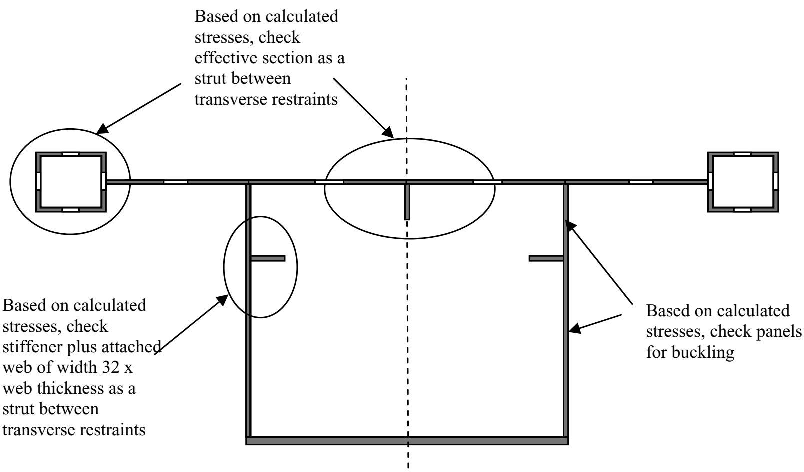


Figure 4. Typical design approach for longitudinally stiffened beam in BS 5400 Part 3

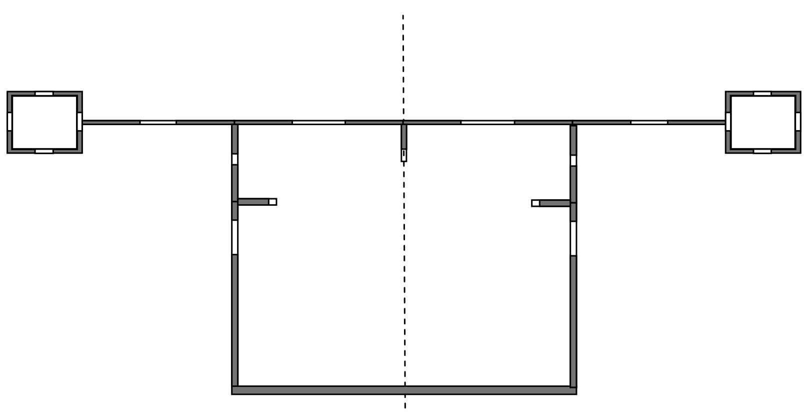


Figure 5. Typical effective cross section for design to EN 1993-1-5

Shear – moment interaction

EN 1993 produces a more economic check of shear and moment interaction than does BS 5400 and consequently has been used in assessment to justify not strengthening existing bridges. It is more economic for three reasons:

- Shear does not interact with lateral torsional buckling resistance. It only interacts with cross section resistance;
- The interaction diagram is a continuous curve, as shown in Figure 6, rather than a series of straight lines as was the case in BS 5400 Part 3;
- Even if the cross section is in Class 3 or 4 (so that the bending resistance is limited to first yield), the interaction is performed using the plastic bending resistance. The interaction is truncated by the requirement to limit the moment to the elastic moment. This has the effect of permitting almost full web shear resistance with full bending resistance, as shown in Figure 6, which reflects the findings of recent non-linear parametric finite element studies.

Longitudinally stiffened cross sections are treated in essentially the same way in EN 1993-1-5 as for unstiffened cross sections, so the same economic benefits can be obtained. To BS5400 Part 3, the check of the cross section would have to be performed on a panel by panel basis in such a way that any shear stress at all has the effect of reducing bending resistance.

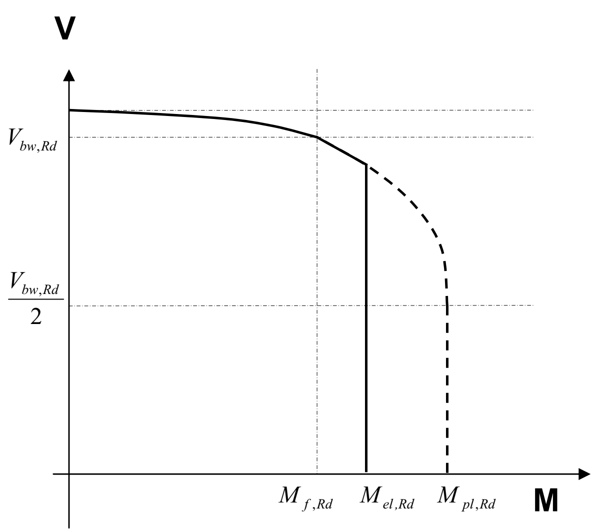


Figure 6. Typical interaction diagram for shear and moment to EN 1993-1-5

Lateral torsional buckling and distortional buckling

Whilst BS 5400 Part 3 gave extensive empirical guidance on lateral torsional buckling, EN 1993 takes a more theoretical approach. EN 1993, as a general approach, gives only an expression for slenderness, $\overline{\lambda}_{LT} = \sqrt{\dfrac{W_y f_y}{M_{cr}}}$, where M_{cr} is the elastic critical buckling moment.

No guidance is given on the calculation of M_{cr}, which tends to lead the designer towards performing a computer elastic critical buckling analysis for its determination. It is not, however, always necessary to do this. For steel-concrete composite members with the deck slab on top of the beams, where buckling is by lateral buckling of the compression flange, a simpler method is provided that avoids the need for this calculation. It is actually simpler to perform than any equivalent check in BS 5400 if the moment does not reverse between restraints. Where there is moment reversal, some interpretation is required, such as that provided in reference 7. As a further alternative, reference 1 provides some methods of calculation without requiring explicit calculation of M_{cr}; this is based on practice to BS 5400 Part 3.

One situation where EN 1993 provides no hand calculation method is for the case of paired beams during concreting, prior to the deck slab providing restraint in plan to the beams. In this situation, recourse can be made to reference 1 or finite element analysis undertaken. The latter will bring rewards, as the bending resistance so derived in accordance with EN 1993 will be significantly greater than that to BS 5400 Part 3. A typical global buckling mode derived from a finite element model is shown in Figure 7. With a bit of experience of creating such models, it can be both much quicker to perform and more economic than the calculation to reference 1, which requires a mixture of hand calculations and plane frame analysis.

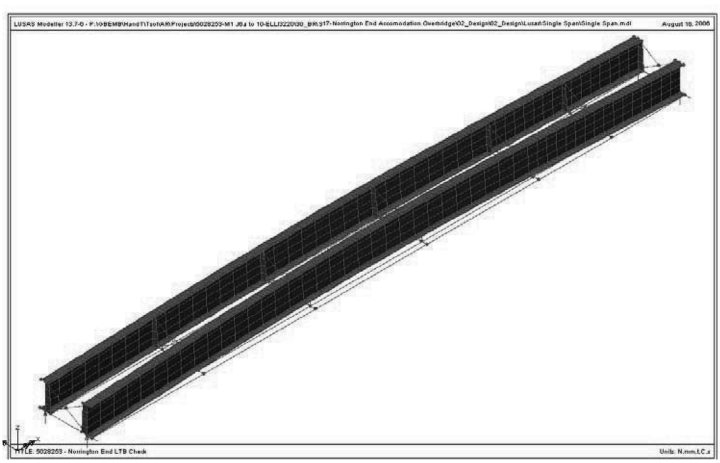


Figure 7. Global buckling mode for paired steel beams during pouring of deck slab

Web transverse stiffeners

The design of web transverse stiffeners provided to enhance shear resistance has provoked some debate in the UK. The original Eurocode proposal was little more than a stiffness requirement. This was augmented, following comments by the UK, to include a strength requirement similar to that provided in BS 5950 (reference 9), but not going as far as the similar strength requirement in BS 5400 Part 3, which was more conservative for asymmetric beams and beams carrying compression. Even with this less conservative proposal adopted in EN 1993-1-5, the Eurocode still provides an effective way of assessing existing structures to demonstrate adequacy where BS 5400 Part 3 suggests inadequacy. The UK National Annex however tightens up the requirements further as discussed in the paper on the NA to EN 1993-1-5.

Non linear analysis

More advanced calculation methods are allowed and codified in EN 1993. Non-linear analysis is one example and its use can lead to significant refinement of designs and can often be used to overcome conservative restrictions in the Eurocode application rules or National Annex provisions. The model in Figure 8 was one of many set up by Atkins on behalf of the Highways Agency to investigate the Eurocode rules for transverse stiffeners resisting shear. The model was set up in accordance with the requirements of EN 1993-1-5 and modelled the geometry of an actual physical test specimen, tested in the 1980s[3]. Not only did the non-linear FE model give results almost identical to the physical test, but it also showed the EN 1993-1-5 rules for stiffeners to be very conservative for this particular beam and the BS5400 Part 3 predictions even more so. Designs incorporating non-linear analysis may, however, be more time consuming to conduct than their elastic analysis counterparts.

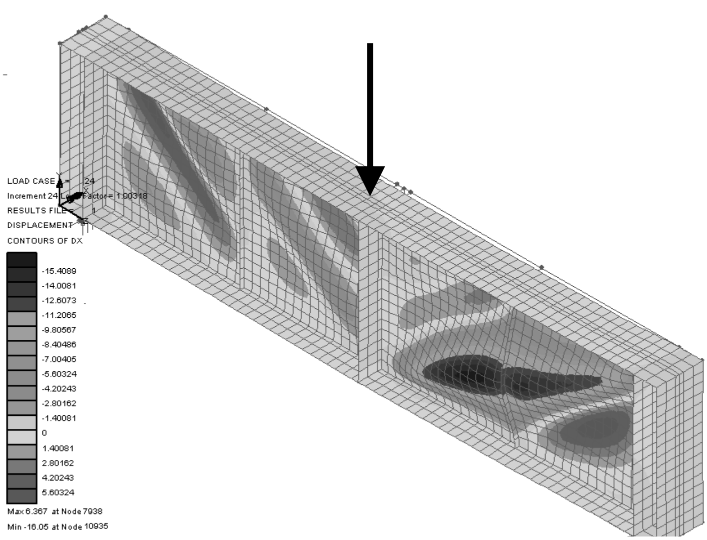


Figure 8. Non-linear modelling of a plate girder in shear

Serviceability

Checks of steel beams at the serviceability limit state are very similar to those currently employed. A new check on web breathing has, however, been introduced to control fatigue in welds at panel boundaries but this check never seems to be a governing one.

For composite bridges, the check of crack widths is significantly different in both approach and outcome and is in many ways more straight forward than in previous practice. Crack widths are calculated under "quasi-permanent" actions (i.e. using the quasi-permanent combination of action). These are essentially permanent loads and the effects of temperature. Having calculated the stress in the reinforcement under these actions, using cracked section properties and adding a correction term to allow for tension stiffening, the resulting stress in the reinforcement is compared against one of two limits; one based on bar spacing and another based on bar diameter, both for a given design crack width, usually 0.3 mm. It is only necessary to satisfy one of the two limits. As a consequence, whereas crack width was often a governing check to BS 5400, it will rarely govern to the Eurocodes.

3 Concluding remarks

There are cultural and technical differences in the Eurocodes that designers will have to get used to and extensive training and guidance documents will be needed in the transition. However, experience to date reveals that designers adapt quickly and there are many similarities to current UK practice. Further motivation for the change should be that the less prescriptive approach and more up-to-date rules provide the designer with greater scope for innovation and economy.

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THE UK NATIONAL ANNEXES TO BS EN 1993-2, BS EN 1993-1-11, AND BS EN 1993-1-12

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Abstract
The National Annex to the Eurocodes is permitted to provide the country specific data and guidance to a number of clauses in Eurocodes which are termed as Nationally Determined Parameters or the NDPs.
This paper gives background information including the bases on which the Nationally Determined parameters for the National Annexes to BS EN 1993-2, BS EN 1993-1-11 and BS EN 1993-1-12 were determined.
In the UK, the Nationally Determined Parameters have been chosen such that the reliability of structures determined using the rules in the Eurocodes and these NDPs is roughly equivalent to that obtained using BS 5400.

Introduction
The National Annex to the Eurocodes is permitted to provide the country specific data and guidance to a number of clauses in Eurocodes which are termed as Nationally Determined Parameters or the NDPs.The objective of this paper is to give the background to the development of the provisions of National Annexes to BS EN 1993-2:2006, BS EN 1993-1-11:2006 and BS EN 1993-1-12:2007. All three National Annexes were prepared by the Working Group WG3 under the auspices of the BSI Subcommittee B/525/10 in conjunction with B/525/31.

In the UK, the Nationally Determined Parameters have been chosen such that the reliability of structures determined using the rules in the Eurocodes and these NDPs is roughly equivalent to that obtained using BS 5400.

The paper gives reasons for the choice of the NDPs for these three Eurocode Parts. While no specific reliability studies were carried out in the selection of the NDPs, comparisons were made against the values determined using safety factors and other requirements contained in BS 5400. Decisions were made about their acceptability by comparing them against the equivalent values in the national standards and also by the calibration studies carried out on behalf of the client authorities namely the Highways Agency and Network Rail.

For convenience the format of the National Annex has been adopted in respect of the heading for the clauses, except that the clause reference has been shortened

UK National Annex to Eurocode 3: Design of Steel Structures – Part 2: Steel Bridges

NA.2.1 Design working life [2.1.3.2(1)]

The design working life referred to by Eurocodes is the period over which a bridge is required to be used for its intended purpose, with maintenance at regular intervals but without major repair being necessary. The design life can be different for temporary structures or structures or components which can be replaced and have a limited life.

The recommended design working life for bridges given in 3-2/2.1.3.2 is 100 years but the National Annex may vary this. The design life quoted in various national standards varies from about 60 years to about 120 years, the latter being specified in BS -5400. It was considered that any reduction from 120 years would be regarded by the public as a reduction in the quality of the UK infrastructure and the National Annex therefore sets the design life at 120 years. The reality however is that the increase (from 100 to 120 years) only influences fatigue calculations and, in some cases, the magnitude of wind and thermal actions; there are no other impacts on the design or construction.

NA.2.2 Durability [2.1.3.3(5)]

The principle that a structure will remain durable during its design working life, assuming appropriate maintenance is carried out, is stated in EN 1990:2004, clause 2.4. Similar requirements were made in the previous British Standards and other standards promulgated by highway authorities such as the Highways Agency. Many factors influence durability including, environmental conditions, quality of workmanship, detailing, protective treatments, maintenance regime and choice of materials.

There is much guidance available on the design of durable bridges, such as the CIRIA Bridge detailing guide[1] and also BD and BA 57 [2, 3]. It was not considered necessary to give further guidance in the National Annex, particularly as this might stifle innovation, but the allowance was given to give additional requirements for particular projects.

NA .2.3 Robustness and structural integrity [2.1.3.4(1)]

The National Annex is permitted to define components which should be designed for accidental design situations. Typical components would be hangers, cables and bearings including other vulnerable components, such as towers or plinths. Since these components would be structure specific and there was a risk of omitting an item if a list were produced, the National Annex simply recommends that these components should be specified for a particular project.

NA.2.4 Robustness and structural integrity [2.1.3.4(2)]

The recommended method for fatigue assessment in the National Annex is the safe life method. The safe life method is intended to provide an acceptable level of reliability without the need for regular inspection for fatigue damage or the need to build in redundancy for failure of a component. This is in line with BS 5400-10[4], where the safe life principle was used.

The use of the damage tolerant method would potentially permit the use of a lower value of partial factor for fatigue resistance. However, this approach requires an inspection regime to be in place that will locate fatigue cracks before they can propagate and also the provision of redundancy in the event of component failure.

Although the damage tolerant method is accepted as an option by the UK National Annex to EN 1993-1-9 it was considered that, for bridges, it was unrealistic to rely upon routine inspection to detect such cracks and hence only the safe life method is recommended in the National Annex to EN 1993-2.

If, for a particular project, there was an overriding requirement to achieve minimum weight, the damage tolerant approach could be specified for that project and this is accepted by the wording of the National Annex clause. In such circumstances a well established maintenance regime should be provided to ensure crack detection during the working life was in place. Further guidance on the damage tolerant method is given in PD 6695-1- 9[5]

NA.2.5 Actions and environmental influences [2.3.1(1)]

The NA is permitted to specify additional actions to those in EN 1991. The actions in EN 1991 are fairly comprehensive and should cater for almost all conceivable design situations. Any additional actions are mostly structure specific and should therefore be given for individual projects.

The action effects for transient conditions such as maintenance and repair are project specific and should be listed in the maintenance manual. These are to be included also for the individual projects.

Fire is generally not considered for the design of bridges.

NA.2.6 Fracture toughness [3.2.3(2)]

EN 1993-1-10 and BS 5400 use different approaches to establish the fracture toughness of a material to prevent brittle fracture. BS 5400-3[6] uses the minimum service temperature to determine the limiting thickness for fracture toughness based on a number of parameters. Whereas Table 2.1 in BS EN 1993-1-10 uses a reference temperature which is based on the lowest air temperature and adjusted for radiation loss, material properties, strain rate and the cold forming; the resulting maximum permitted thickness depends on this reference temperature. However, UK view was that the EN 1993-1-10 rules did not take a number of other factors satisfactorily into consideration, including gross stress concentrations, detail types and residual stresses and did not provide sufficient guidance.

As a result, the recommendation in the National Annex is not to follow the example in Table 3.1 of EN 1993-2 but to follow recommendations in the National Annex to BS EN 1993-1-10 which, together with the PD 6695-1-10, provide the necessary guidance. The resulting thickness for a certain grade is approximately equal to the previous practice in the UK.

NA.2.7 Fracture toughness [3.2.3(3)

EN 1993-2 recommends that, for parts in compression, a value of $\sigma_{Ed}/f_y = 0.25$ (i.e. a tensile stress level of 25% of yield) is used to determine toughness requirements. EN 1993-1-10 makes no similar requirement. At present the National Annex wrongly adopts the recommendation of EN 1993-2.

However for compression elements, particularly slender components, consideration of tensile stress much greater than the above is needed to account for the residual tensile stress, lack of fit and buckling stresses. The UK National Annex should, therefore, be modified to state that for parts in compression the recommended value in EN 1993-2 should not be used. Instead the recommendation given in the National Annex to BS EN 1993-1-10- including the guidance in PD 6695-1-10 should be followed.

NA.2.8 Through thickness properties [3.2.4(1)]

Improved through thickness properties may sometimes be required to prevent lamellar tearing. The main risk of lamellar tearing occurs with cruciform joints, corner joints and with full penetration butt welds where the thicker plate is welded to the thinner one, thus maximising the through thickness forces. The approach in the UK to date has been to detail to minimise such risk through thickness testing. The fabricator would then use welding methods to minimise the risk of lamellar tearing and test the joint ultrasonically afterwards to ensure cracks had not formed.

The above approach has been adopted in the National Annex to BS EN 1993-1-10 and therefore the National Annex to BS EN 1993-2 rejects the example in 3-2/Table 3.2 and directs the designer to 3-1-10/2.2.

NA.2.9 Cables and other tension elements [3.4(1)]

No particular types of cable are specified in the National Annex, which simply leaves the types to be specified for the project. See further comment on the National Annex to BS EN 1993-1-11 below.

NA.2.10 Bearings [3.5(1)]

No specific recommendations are given in the National Annex on the choice of bearings which leaves the types to be specified for the project. Bearings are covered by EN 1337 and guidance on that Standard is given in PD 6703:2009 [7].

NA.2.11 Other bridge components [3.6(1) and (2)]

There are design standards and guidance documents available in the UK for the design of parapets, expansion joints, deck waterproofing and other ancillary items. Some of these documents contain client-specific requirements.

No particular types of component are specified in the National Annex, which simply leaves the types to be specified for the individual project.

NA.2.13 Durability [4(1)]

Adequate provision for access should be made to all bridges at the design stage so that inspection and maintenance can be carried out efficiently. Guidance on access is given in the CIRIA Bridge detailing guide and also BD and BA 57 produced by the Highways Agency.

Since access requirements would be bridge specific, the UK National Annex requires these to be specified for the individual projects. Additionally, the National Annex gives requirements for box girders with internal access in order that that they should not allow a build up of moisture (through suitable drainage of the void); and that they should be well ventilated (to minimise the risk of a build up of harmful gases).

NA.2.14 Durability [4(4)]

The National Annex is permitted to give guidance on sealing against corrosion, air tightness of box girders and the provision of extra steel thickness for inaccessible surfaces.

The measures to be taken and additional sacrificial thicknesses given are primarily based on those in BS 5400-3. They have been modified with due consideration of ISO 9223[8] and BS EN ISO 12944-2[9].

The important consideration is that all structures should be inspectable for reasons of durability with provisions of access. In exceptional circumstances where access is not provided and girders are effectively sealed against the ingress of moisture and contaminants, additional thickness should be provided.

From experiences it has been found that moisture could accumulate in a box section or water could even ingress in the section which has been sealed by welding. To prevent corrosion and to prevent bursting of the section due to ice formation, drainage should always be provided in a closed section.

It may be worth noting that this is the only place where sacrificial allowances are made for weathering steel and thus they apply whether there is facility for access or not.

NA.2.15 Effects of deformed geometry of the structure [5.2.1(4)]

Second order analysis needs to be used where second order effects are significant and it is therefore the default method of analysis in the Eurocode. According to 3-1-1/5.2.1, however, first order analysis may be used if the increase of the relevant internal moments or forces or the structural behaviour due to deformations are negligible i.e. less than 10%. 3-2/5.2.1 states that this condition is met if α_{cr}, which is the factor by which the design loading would have to be increased to cause elastic instability in a global mode, does not exceed a value of 10 for elastic analysis. The value of α_{cr} will usually be obtained by computer analysis but hand calculation methods to derive critical load factors could also be employed for some structures.

No guidance on the definition and calculation of α_{cr} is provided in the National Annex, as there is much available industry guidance covering this subject, such as reference 10.

NA.2.16 Methods of analysis considering material non-linearities [5.4.1(1)]

In general elastic methods of global analysis should be used for bridges to determine internal forces and moments. However for accidental design situations such as impact on bridge parapets, piers or decks, plastic global analysis might be appropriate.

No specific guidance is given in the National Annex as to when plastic analysis may be used; it is left to be specified for the particular project.

NA.2.17 Ultimate limit states – Partial factors [6.1(1)]

The recommended values of the partial factors for resistance to be used for bridges are given in 3-2/6.1. These values were in many cases arrived at through statistical analysis to achieve a target reliability index (as defined in EN 1990) of 3.8.

Resistances derived using Eurocode rules and the recommended partial factors were compared to those derived from BS 5400-3. Generally the comparison was close. However, there are some areas where the Eurocode rules provide some increase in resistance due to improved understanding of behaviour of materials.

The National Annex accepts all the recommended values

NA.2.18 Shear lag effects [6.2.2.3(1)]

The National Annex directs that the National Annex to EN1993-1-5 should be followed. The paper[11] on the UK National Annex to EN 1993-1-5 provides the necessary background.

NA. 2.19 Effects of local buckling for class 4 cross sections [6.2.2.5(1)]

The National Annex permits both methods of calculation in EN 1993-1-5 (Section 4 and Section 10). The paper on the UK National Annex to EN 1993-1-5 provides background, together with more information on the methods.

NA.2.20 Lateral torsional buckling curves for rolled sections or equivalent welded sections [6.3.2.3 (1)]

The buckling curves given in 3-1-1/6.3.2.3 have a longer plateau length (to a slenderness of 0.4) than the curves for the general case (which have the same plateau length of 0.2 as the strut curves). 3-1-1/6.3.2.3 allows the National Annex to modify this but the National Annex to BS EN 1993-1-1 confirms the length and the value of the β parameter; it also defines validity in relation to *h/b* ratio. Clause 3-2/6.3.2.3 permits further information to be given but the NA to BS EN 1993-2 refers to PD 6695-2 for general guidance on lateral torsional buckling. The objective is to enable UK designers to use an alternative tried and tested method which is based on BS5400-3.

The reference to “equivalent welded sections” in Eurocode is understood to refer welded members of the same size as available rolled sections.

NA.2.21 General method for lateral and lateral torsional buckling of structural components [6.3.4.2(1)]

3-2/6.3.4.2 (1) refers to EN 1993 clause 6.3.2.4(1) which is marked (B) for use on buildings only. The inclusion of clauses 6.3.2.4(1) to (3) for bridges has been debated for a considerable time at the CEN committee and eventually the conclusion was to include these as the NDP. These clauses offer a simplified method which restricts the values of the buckling curve parameters. The National Annex to BS EN 1993-2 refers to PD 6695-2 for general guidance on lateral torsional buckling to enable UK designers to use the method in the PD as an alternative.

In practice, the modified method given by 3-2/6.3.4.2 (1) is essentially the same as the strut model covered more comprehensively in 3-2/6.3.4.2 (2) to (7).

NA.2.22 General method for lateral and lateral torsional buckling of structural components [6.3.4.2(7)]

3-2/6.3.4.2 (7) offers a method of allowing for variation in force along the length of the simplified strut. It is a method that will be very useful for hogging moment regions of beam and slab bridges.

The method is similar to that for U frames given in BS 5400-3 but usefully allows the beneficial variation in bending moment to be considered in the calculation via the expression for "m". These expressions given in EN 1993-2 were derived from finite element analyses and represent best-fit approximations to the data obtained. The resulting method gives reasonable agreement with previous practice to BS 5400-3 and hence the National Annex to BS EN 1993-2 offers no modification of the expressions for m.

NA.2.23 Serviceability limit states [7.1(5)]

Although 3-2/7.1 allows the National Annex to give guidance on serviceability requirements for certain types of bridges, the UK National Annex confines itself to the recommendation that additional requirements should be specified for individual projects.

NA.2.24 Limitations for stress [7.3(1)]

The recommended value for the partial factor $\gamma_{M,ser} = 1.0$ is adopted. This is consistent with previous practice in BS 5400-3.

NA.2.25 Injection bolts [8.1.3.2.1(1)]

Injection bolts are bolts in which the cavity produced by the clearance between the bolt and the wall of the hole is completely filled up with a two component resin. Such bolts were not covered by BS 5400, presumably because there was not sufficient confidence in the durability of the resin at the time of the preparation of BS 5400. Consequently there is hardly any experience of the use of bolts in this country for bridges. The National Annex to BS EN 1993-2 simply states that the requirements for their use should be specified for a particular project.

NA.2.26 Hybrid connections [8.1.6.3(1)]

BS EN 1993-1-8 generally requires that the resistance of joints made with different connectors (e.g. welds and bolts) be based on the strength of the stiffer connectors. This is because there may be inadequate ductility in the stiffer connectors to maintain their peak load while subsequent load is shed to the other connectors. In the case of hybrid connections involving preloaded bolts designed not to slip at ULS and welds, it is implicit in 3-1-8/3.9.3 that both connections are equally stiff and can therefore share load equally. The UK National Annex invokes caution (from the rule in BS 5400-3) and limits the total resistance to 90% of the combined strengths in recognition that the load sharing may still not be able to mobilise the full resistance of each connection.

NA.2.27 Butt welds [8.2.1.4(1)]

The provisions for butt welds in EN 1993-1-8 are straightforward and similar to those in BS 5400-3. No additional guidance was considered necessary for the UK.

NA.2.28 Plug welds [8.2.1.5(1)]

Plug welds are not generally used for new bridge designs in the UK. However design requirements are given in 3-1-8/4.3.5 and 4.8 which are broadly comparable to those in BS 5400-3. The committee felt that any further requirements would be project specific. Although Eurocode permits plug welds for use in shear, their use is not recommended for major structural connections in bridges.

NA. 2.29 Flare groove welds [8.2.1.6(1)]

There is no experience on the use of flare groove welds for bridge applications in the UK. However the design throat thickness of such welds is given in 3-1-8/4.3.6 and it is understood that they can be designed in the same way as partial penetration welds. The National Annex to BS EN 1993-2 simply states that (additional) requirements should be specified for the individual projects.

NA.2.30 Eccentrically loaded single fillet or single-sided partial penetration butt welds [8.2.10(1)]

The design of eccentrically loaded welds is covered in 3-1-8/4.12. The National Annex to BS EN 1993-2 says that they should not be used to transmit bending moments about the longitudinal axis of a weld; this effectively prohibits the situations in (a) of 3-1-8/Figure 4.9 and is consistent with BS 5400-3. The situation in (b) of the Figure is not ruled out by the National Annex, it simply says that the use (of these welds) for other situations (i.e. other than in Figure 4.9(a)) should be specified for a particular project.

NA.2.31 Analysis of structural joints connecting H- and I-sections [8.2.13(1)]

BS EN 1993-1-8 provides comprehensive requirements for the design of structural joints for H and I sections. Such joints are used mainly for buildings. Clause 3-2/8.2.13 allows the National Annex to give further guidance on their use. No guidance is offered in the National Annex in relation to their use in bridges; it simply states that the requirements should be specified for a particular project.

NA.2.32 Hollow section joints [8.2.14(1)]

Hollow section joints are not covered in BS 5400 whereas the requirements in BS EN 1993-1-8 section 7 are comprehensive. No guidance is offered in the National Annex to BS EN

1993-2 in relation to their use in bridges; it simply states that the requirements should be specified for a particular project.

NA.2.33 Design of road bridges for fatigue [9.1.2(1)]

The National Annex to BS EN 1993-2 recommends that, in line with current practice in the UK, all details that are subjected to cyclic loading should be checked for fatigue. It was considered unnecessary to specify situations where a check was not required.

NA.2.34 Design of railway bridges for fatigue [9.1.3 (1)]

The recommendations are the same as in NA.2.33 for road bridges.

NA.2.35 Partial factors for fatigue verifications [9.3(1)]

The value of the partial factor for fatigue loads recommended in 3-2/9.3.1 is adopted by the National Annex to BS EN 1993-2. The value (1.0) is the same as that specified in BS 5400. The acceptance of this value was made after calibration studies using this value and the recommended value of the partial factor for fatigue resistance against assessments in accordance with BS 5400. Reasonable correlation was obtained in these calibration studies.

NA.2.36 Partial factors for fatigue verifications [9.3(2)]

The value of the partial factor for fatigue resistance recommended in 3-2/9.3.2 is adopted. This is consistent with the National Annex to BS EN 1993-1-9, which adopts a value of 1.1 irrespective of the consequence class. See comment on NA.2.35.

NA.2.37 Fatigue stress range [9.4.1 (6)]

The main load model given in BS EN 1991-2 has been calibrated against continental highway traffic. Factors have been introduced to take into account heavier or lighter traffic based on calibration. The fatigue assessment method using λ factor are based on fatigue load model 3 of BS EN 1991-2 and is analogous to the simplified procedure for checking railway and highway bridges in BS 5400-10. The basic idea of this method in the Eurocode is to relate the damage induced by the stress spectrum to an equivalent stress corresponding to 2x 10^6 cycles. From a wide set of data obtained from insitu site measurements of road traffic loads, it has been possible to define the representative values of λ needed for the design of bridges. The procedures are conservative due to the need to ensure that this does not give rise to unsafe designs. Where a more realistic fatigue load model is required guidance given in Annex A of BS EN 1993-1-9 together with the UK National Annex and PD 6695-1-9 should be followed. Basically it points designers to using fatigue load model 4 as specified in the National Annex to BS EN 1991-2. This is similar to the approach used in the UK for assessing fatigue.

NA.2.38 Damage equivalence factors λ for road bridges [9.5.2 (2)]

The λ_1 factors for the effect of influence line length have been derived from parametric studies. The National Annex does not give any additional information.

NA.2.39 Clause 9.5.2 (3) Damage equivalence factors λ for road bridges

The λ_2 depends on the annual lorry flow and on traffic composition. The expression is calibrated against the traffic at Auxerre in France which is known to be more heavily congested compared to any other places in Europe.

The Qm1 value of 260 kN in the National Annex was derived from UK Department of Transport road vehicle census data from a large number of permanent weigh-in-motion sites on motorways and major roads across the UK in the year 2000.

NA.2.40 Damage equivalence factors λ for road bridges [5.2 (5)]

The clause simply reiterates the recommended value of the design life specified in NA.2.1 (120 years). For this design life, expression (9.11) then gives a value of $\lambda_3 = 1.037$.

NA.2.41 Damage equivalence factors λ for road bridges [9.5.2 (6)]

λ_4 is defined as the factor to account for the traffic in other lanes. The study carried out in the UK on combined damage from more than one lane appeared to show little additional fatigue from the theoretically bigger stress cycles from joint loading, so these N_j values can be based on relative flow numbers. For current UK purposes, without resorting to further research, it was agreed that there is no justification in using different 'average' lorry weights in each lane, so the lane Q_j values can be taken as 260kN for all lanes. In view of the conservatism of the method, this approach is considered satisfactory.

NA.2.42 Damage equivalence factors λ for road bridges [9.5.2 (7)]

λ_{max} is the limiting value of λ taking into account of the fatigue limit. It has not been possible to verify the validity of this as the background information was not available. However calibration studies have shown that the results correlate reasonably well with BS 5400-10.

NA.2.43 Damage equivalence factors λ for railway bridges [9.5.3 (2)]

Recommended values should be used except that for specialised lines that are not covered by the code λ_1 should be project specific.

NA.2.44 Fatigue strength [9.6 (1)]

No additional information is given in the National Annex. The information in EN 1993-1-9 is comprehensive and it was deemed unnecessary to exclude any of the details in EN 1993-1-9.

NA.2.45 Post weld treatment [9.7(1)]

The beneficial effects of post-weld treatment are well documented but are not covered by EN 1993-2 or EN 1993-1-9, nor were they covered in BS 5400-10. Toe grinding will raise the relevant S-N curve by 30% according to BS 7608[12]. It is important to note that BS 7608 recommends that benefits of improvement techniques should be taken into account when the need to fatigue strength is discovered at a late stage of fabrication or when the structure is already in service. It is difficult to quantify the extent of increase unless trials are carried out for such techniques for each detail and thus it was not considered realistic to provide any additional guidance. Any additional information should be project specific, based on research evidence.

NA.2.46 Anchorage of bearings [A.3.3 (1)]

In the absence of previous design rules (in BS 5400) for summing the resistances from friction and mechanical anchorages, the default recommendations for material factors on friction were accepted in the UK National Annex to EN 1993-2. However, paragraphs (2) and (3) of 3-2/A.3.3 should always be observed. These state that in calculating frictional resistance, the dynamic effects of traffic should be considered for highway bridges and friction should not be utilised for rail bridges or in seismic situations. The practical consequence of these provisions is that it would usually be expedient to design the ULS resistance based on the anchorage bolts only rather than try to estimate the minimum bearing reaction available for friction under dynamic effects. There was no provision to state this conclusion in the UK National Annex.

NA.2.47 Resistance to rolling and sliding [A.3.6 (2)]

Although it was not possible to discover the background to the rules and value of α in 3-2/A.3.6, the value is consistent with that in BS EN 1337-1:2000. The recommended value is adopted.

NA.2.48 Determination of design values of actions on the bearings and movements of the bearings [A.4.2.1 (2)]

The approach to bearing movement calculation given in 3-2/A.4.2 differs from the practice in BS 5400, in that all uncertainties in bearing positions are catered for by additional shifts in temperature range relating to the final position . The movements that can occur during construction from moving the point of fixity can therefore be directly estimated. Also a characteristic temperature range is used with a further shift to achieve a design value rather than the application of a gamma factor to the characteristic value. Creep and shrinkage is not included via the temperature shifts and would need to be added separately in concrete bridges.

There is an inconsistency between EN 1991-1-5 and EN 1993-2 Annex A which has been addressed. Basically the designer should obtain the values of the initial bridge temperature T_0 and the characteristic value of the maximum expansion and contraction ranges from EN 1991-1-5 (and the UK National Annex for design in the UK) and the adjustments are made using the value of ΔT_0 given by 3-2/A.4.2 and the UK National Annex.

A comparison study was carried out between the current practice in the UK and the proposed new practice to BS EN 1993-2 Annex A rules which show reasonable correlation.

NA.2.49 Determination of design values of actions on the bearings and movements of the bearings [A.4.2.1 (3)]

The recommended values of ΔTo given in Table A.4 provide reasonable agreement with existing practice and were thus adopted.

NA.2.50 Determination of design values of actions on the bearings and movements of the bearings [A.4.2.1 (4)]

Based on a comparison study carried out on selected example bridges, the recommended values of ΔT_0 given in Table A.4 and a value of 5 degrees C for $\Delta T\gamma$ were adopted in the UK National Annex. However, it should be noted that the temperature difference ΔT_k should be taken as the maximum expansion range or the contraction range as appropriate according to BS EN 1991-1-5, as modified by the UK National Annex. The definition is different from that given in BS EN 1993-2. Note that the value ΔT_d^* is a design value and therefore does not require the application of a partial factor, even though its component ΔT_k is a characteristic value.

NA.2.51 Actions for accidental design situations [A.4.2.4 (2)]
Reference has been made to the National Annex to EN 1991-1-7 for additional information.

NA.2.52 Highway bridges [C.1.1 (2)]
Surfacing of orthotropic decks is a specialist activity. No additional technical information is provided.

NA.2.53 Thickness of deck plates and minimum stiffness of stiffeners [C.1.2.2 (1)]
Although there are no previous British Standards for the design of orthotropic bridges, practice has been to adopt certain minimum design thicknesses for deck plates and stiffeners for the design of new orthotropic bridges. The values recommended in 3-2/C.1.2.2 are greater than those used in the UK. For example, the thickness of the plates used on most existing orthotropic boxes in the UK is 12 mm, with a surfacing thickness of 40 mm. The increased thicknesses of plates and surfacing have been driven by the need to prevent cracking of the deck plate and joints with stiffeners. This problem of weld cracking has been exacerbated over the years as the lorry weights tend to get heavier.

The UK National Annex accepts the recommended values of deck plate thickness for carriageways in 3-2/C.1.2.2, but with a reduction factor for trough spacing less than 300 mm and a permitted further reduction of 2 mm for lanes occupied by the light vehicles. The maximum clear spacing of troughs was also increased to 380 mm.

For footbridges, a distinction was made between the parts which carry maintenance vehicles and those which do not. The National Annex recommendations are in line with the Steel Bridge Group Guidance Notes [13].

A.2.54 Thickness of deck plates and minimum stiffness of stiffeners [C.1.2.2 (2)]
The minimum stiffness requirements fit existing practice. No additional information is provided.

NA.2.55 Combination of effects from local wheel and tyre loads and from global traffic loads on road bridges [E.2 (1)]
It has not been possible to establish the background to this requirement and hence the more cautious approach of setting the combination factor to 1.0 for all span lengths was adopted.

NA.3 Decision on the status of the informative annexes
Reference to Annexes A and B is missing from the UK National Annex to BS EN 1993-2 as they were only changed to informative status at a very late stage. The Annexes are due to be moved to EN 1990 at a future date.

The National Annex will need to be corrected to include a reference to Annexes A and B stating that they may be used.

The National Annex presently states that Annexes C, D and E may be used.

UK National Annex to Eurocode 3: Design of Steel Structures – Part 1-11: Design of Structures With Tension Components

BS EN 1993-1-11 covers cables which can be adjusted or replaced. Its primary coverage is the design of the tensioned elements of bridges, including the design of wires, cables, anchorages and saddles rather than the design of the overall bridge. The implicit assumption is that the design of the decks, towers etc will be carried out using other Eurocodes. It does not cover cables for suspension bridges (other than in passing) or the design of externally post tensioned bridges.

The design requirements of cables for bridges are not covered in BS 5400. Many varieties of tension elements are currently used in bridge design. The choice is dictated by cost, ease of maintenance and aesthetics. Spiral strand systems are comparatively easy to inspect and maintain. Locked coil strand although more expensive are smooth in appearance and claim to have superior corrosion properties. Tension bars such as Macalloy bars are often used for footbridges or for smaller applications. Since the types of cables being used for bridges depend, among other factors, on the durability, environmental conditions, ease of maintenance and fatigue behaviour no specific recommendations were given in the National Annex on which to choose. EN 1993-1-11 now provides much guidance on such elements.

NA.2.1 Replacement and loss of tension components [2.3.6]

(1) NOTE

EN 1993-1-11 requires the replacement of at least one tensioned element to be considered in the design of a bridge but does not prescribe the transient conditions this to be undertaken in. With increasing congestion in the network there is a pressing need to keep the bridges open under all circumstances. Hence the National Annex recommends that, during replacement, all elements of the structure should satisfy the relevant serviceability and ultimate limit state requirements without any restrictions to traffic or other imposed loads, except where agreed otherwise for the individual projects. The default is therefore that the bridge should satisfy all the normal checks required at ULS and SLS while a tension element is being replaced.

(2) NOTE

EN 1993-1-11 is not prescriptive about consideration of accidental loss of tension components, except saying that it should be taken into account where required. The UK National Annex makes the recommendation that, unless otherwise specified, the loss of any one hanger or stay be considered in the accidental combination.

Where such loss cannot be designed for, the National Annex recommends that protective measures should be specified.

The primary cause of damage that may arise is due to vehicle impact, for which the protection measures could include provision of high containment parapets. It is recognised that it is not feasible to design against damages arising from effects such as aircraft strikes or most terrorist attacks.

The requirements for the accidental loss of one cable are identical to that specified in the PTI's recommendations[14] for cables and also in the SETRA publication[15].

NA 2.2 Transient design situation during the construction phase [2.4.1(1)]

In the absence of any previous guidance on partial factors for the transient design situation (in BS 5400), the recommended values in BS EN 1993-1-1 were adopted.

NA 2.3 Strengths of steels and wires [3.1(1)]

There is very little evidence to support the need to impose a maximum value for f_u for durability reasons. Indeed, such a limit could hinder the development of wires with strength in excess of 2000 N/mm^2. The National Annex removes the limit, although noting that research might find that such steels are more susceptible to premature failure. The National Annex permits the use of higher strengths, where agreed for the individual projects.

NA 2.4 Corrosion protection of the exterior of Group B tension components [4.4(1)]

It was considered unnecessary to prescribe grades of stainless steel for cables except for special situations. The UK NA recommends that the protection should be specified for the individual projects.

NA 2.5 Corrosion protection of Group C tension components [4.5(4)]

Concerns have been expressed about the use of grease at high temperature and hence the UK National Annex recommends that trials should be carried out before accepting any of the soft fillers. Cement grout has low tensile strength which could crack due to movement of the cable allowing water and moisture penetration causing corrosion and fretting of the strands. Hence cement grout is not permitted.

NA 2.6 Transient construction phase [5.2(3)]

The recommended values for γ_P were adopted.

NA 2.7 Persistent design situation during service [5.3(2)]

For some structural types, the combination of P and G actions into a single action (G+P) is not appropriate because normal site monitoring of deflections and adjustment of cables will be insufficient to detect if there is an unintended imbalance between G and P. The view that P and G should not be combined into a single action for such structures was made by the UK during drafting of EN 1993-1-11 but no specific guidance was incorporated in EN 1993-1-11. To ensure that G and P are not treated inappropriately as a single action for such structures, the UK National Annex states that the project specification should give the values of partial factors that are to be used.

NA 2.8 Prestressing bars and Group B and C components [6.2(2)]

The proposed reduction factor on partial material factor of 10% where measures are taken to suppress bending moments affecting the cable in the anchorage zone is in line with current international practice and is the same as proposed in reference 15. The recommended values have therefore been adopted.

NA 2.9 Slipping of cables over saddles [6.3.2(1)]

The recommended value of the partial factor for friction of 1.65 was considered acceptable. The requirement in reference 15 is similar, being either 1.5 or 2.0 depending on whether the friction value used is test based or not.

NA 2.10 Design of saddles [6.3.4(1)]

The recommended value of a cable force of 10% greater than the breaking strength of the cable for the design of the saddle seems reasonable and is adopted.

NA 2.11 Slipping of cables over saddles [6.4.1(1) P]

The recommended value for $\gamma_{M,fr}$ of 1.65 seems reasonable and is adopted.

NA 2.12 Stress limits [7.2(2)]

The recommended values for f_{const} in Table 7.1 and f_{SLS} in Table 7.2 are in line with current international practice for the design of cable stayed bridges and are adopted.

NA 2.13 Waterproofing [A.4.5.1 (1)]

There are no equivalent test requirements in existing UK practice. The test method for water tightness given in the UK National Annex has been taken from reference 15.

NA 2.14 Corrosion protection barriers [A.4.2]

The test requirements for different void fillers and the properties have been largely taken from reference 15.

UK National Annex to Eurocode 3: Design of Steel Structures – Part 1-12: Additional Rules for the Extension of EN 1993 up to Steel Grades S700

NA.2.1 Additional rules to BS EN 1993-1-1

NA.2.1.1 – 2.1 (3.1(2))

Tables 1 and 2 in respect of the strengths of higher grade steel are satisfactory and can be used for bridgework provided the other requirements in BS EN 1993-1-12 are followed.

Over recent years, there has been significant development in the application of higher strength steel particularly in the USA, Sweden and Japan. There are areas in bridgework where higher strengths steel can be used efficiently for example in the design of lighting columns, bridge parapets, sign and signal gantries where the higher fracture toughness and the reduced steel weight can be beneficial. Its application to long span bridges can also be considered in specific situations. While further developments are necessary for wider application of higher strength steel particularly in respect of improved fabrication techniques, the use in hybrid construction etc there is no reason why its use should not be encouraged.

NA.2.1.2 – 2.1 (3.2.2 (1))

There has been considerable debate over the choice of limiting ration f_u/f_y ratio, which 3-1-12/3.2.2 recommends, should be 1.05. This is less than the value of 1.1 for steel grades up to S460 3-1-1/3.2.2 (which is accepted by the UK National Annex to EN 1993-1-1, except for when plastic global analysis is used). However, there is little evidence to suggest that this ratio affects ductility, so it is only the reserve of strength beyond yield which is questioned and whether 'strain hardening' is implicitly invoked in the application rules.

For shear for example, the factor η (which caters for strain hardening) given in 3-1-5/5.1 is 1.0 for steels in excess of 460 MPa, so no benefit is taken from the reserve.

A survey of existing steels used in USA, Japan and Europe showed that a minimum f_u/f_y ratio in excess of 1.1 was generally achieved. Thus the UK National Annex takes the opportunity to be cautious and adopts 1.1 as a minimum value.

The recommended minimum elongation at failure of 10 % was increased to 15% in line with that for steels grades up to S460 in 3-1-1/3.2.2 (which is also accepted by the UK National Annex to EN 1993-1-1, except when plastic global analysis is used). This provides greater ductility.

NA.2.1.3 – 2.1 (5.4.3(1) Note)
Plastic analysis is not permitted for high strength steels as there is inadequate confidence in the rotation capacity.

NA.2.1.4 – 2.1 (6.2.3(2) Note)
The recommended value of γ_{M12} is adopted (the value is the same as that for γ_{M2} in 3-1-1/6.1, which is adopted by the National Annex to BS EN 1993-1-8).

NA.2.2 Additional rules to BS EN 1993-1-8

NA.2.2.1 - 2.8 (4.2(2) Note)
There is little experience in the UK of welding joints with under matched electrodes but since they are being used in Europe their use is not ruled out. The UK National Annex recommends that the strength class of electrodes should be specified for the individual projects.

NA.2.3 Additional rules to application parts BS EN 1993-2 to BS EN 1993-6

NA.2.1.1 - 3.1 ((1) Note)
No additional limits are recommended.

The use of higher grades steel is likely to be subject to greater scrutiny than for grades up to S460. This is primarily because there is not a great deal of experience in the UK for the use of such steel.

Acknowledgements
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EN 1993-1-5: THE UK NA FOR EN 1993-1-5

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Abstract
The objective of this paper is to give the background to the development of the National Annex to BS EN 1993-1-5, focussing particularly on those aspects which relate to the design of bridges. In producing the National Annex, the relevant clauses in the Eurocodes were reviewed, comparing them against clauses in the UK bridges standard BS 5400 and other published national and international standards. Where significant discrepancy with existing practice was revealed, further justification was sought from other research material and the Eurocode clauses were then accepted or amended to suit.

Introduction
The objective of this paper is to give the background to the development of the National Annex to BS EN 1993-1-5, focussing particularly on those aspects which relate to the design of bridges. In producing the National Annex, the relevant clauses in the Eurocodes were reviewed, comparing them against clauses in the UK bridges standard BS 5400 and other published national and international standards. Where significant discrepancy with existing practice was revealed, further justification was sought from other research material and the Eurocode clauses were then accepted or amended to suit.

The remainder of this paper looks at each National Annex entry in turn and provides explanation for the particular requirements given. References to clauses in EN 1993-1-5 have been abbreviated below. For example, 3-1-5/3.3(1) is a reference to clause 3.3(1) of EN 1993-1-5.

NA.2.1 Effective Width Models for Global Analysis [2.2(5)]
Both the loss of stiffness and the reduction in ultimate strength in a plate due to local buckling (caused by the non-uniform axial stress and also out of plane bending stresses due to the deflections in Figure 1) can be modelled using effective widths for the plate panels. The loss of stiffness due to plate buckling is less marked than the loss of strength and hence the widths appropriate to global analysis and to section resistance calculation are not the same with the latter being smaller; the secant stiffness is appropriate for section stiffness calculation whereas the tangent stiffness is relevant to strength calculation.

In BS5400 Part 3[1], the effect of local buckling was neglected altogether in global analysis. In Eurocode 3, local buckling must be considered in global analysis, but only where it results in an effective width for ULS section design lower than 50% of the actual width. Since the reduction in stiffness will be less than the reduction in strength used in the calculations, the recommended limiting value of $\rho = 0.5$ was considered a reasonable cut-off for the reduction in plate strength above which the effect of plate buckling on stiffness need not be considered in the global analysis. In practice, this limit should ensure that plate buckling effects rarely need to be considered in global analysis to EC3.

To explore the validity of this limit, the global bending moments in finite element models of fixed ended beams of uniform depths of 2m under uniformly distributed loading were analysed. The reduction factor at the supports ρ_0 was taken as 1.0 and 0.5, modified for intermediate elements by varying $\lambda_{p,red}$ with the square root of the bending moment. The compression flanges at the supports were sized to provide the assumed value of ρ. The greatest adverse disparity between the calculated moment at the support assuming uniform properties and that allowing for effective widths was no greater than 7.8% for $\rho_0 = 0.5$ when taking the thickness of the compression flange such as to give the same utilisation at mid-span as at the supports.

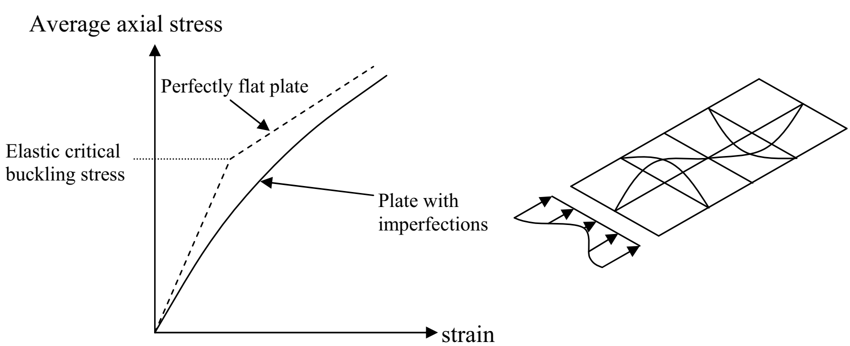

Figure 1. Deflection and stress-strain behaviour for a buckling plate

NA.2.2 Shear Lag at the Ultimate Limit State [3.3(1)]

At ULS, the effective width for shear lag is much greater than at SLS due to a certain amount of plastic redistribution. In previous UK practice to BS5400 Part 3, shear lag was considered in section analysis at SLS, but not at ULS. The choice of method in the National Annex was therefore strongly influenced by the desire to reduce the effects of shear lag at the ULS as far as possible. The method in 3-1-5/3.3(1) NOTE 3 (elastic-plastic behaviour) was thus chosen due to its considerations of plasticity and its provision of the greatest effective width of the three methods permitted as shown by comparisons for a wide range of values of b_0/L_e and buckling factors ρ.

Figure 2 shows the difference in effective width produced at SLS (elastic behaviour) and ULS (elastic-plastic behaviour) for a beam without longitudinal stiffeners at an internal support. The difference is considerable and shows that effective widths at ULS will often be very close to the full gross width. To achieve this redistribution, it is therefore vital that the various parts of the steel cross section possess enough ductility to allow this redistribution to occur without buckling. Torsional buckling of stiffeners, which is a brittle mode of buckling, must therefore be prevented and this is achieved through complying with clause 9.1 of EN 1993-1-5.

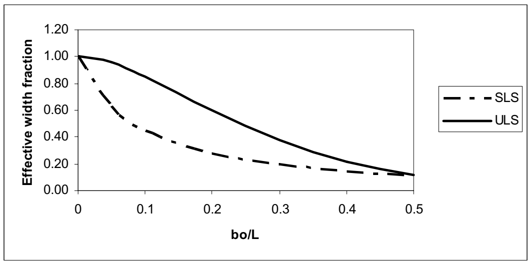

Figure 2. Effective widths for shear lag for different b_o/L ratios

NA.2.3 Effective Cross Section [4.3(6)]

Clause 4.3(6)a) of EN 1993-1-5 requires the increase in the flange stress in hybrid girders to be taken into account by limiting the web stress to the web yield stress. This implies that the section modulus should be calculated on a partially plastic basis with a cut-off in web bending stress with a fraction of the moment assumed to be shed to the flanges. Analysis of a range of unstiffened beams with different ratios of flange area/web area down to 0.5 and flange yield stress/web yield stress up to the practical limit of 2.0 has shown no more than 30% of the web moment to be shed to the flanges.

Although BS5400:Part 3 does not allow load shedding from unstiffened webs, limited finite element analysis by Harding et al[2,3] showed that, even for webs with unrestrained boundaries, load shedding from the web of 30% is admissible.

Clause 4.3(6)b) requires the effective area of the web to be based on the flange yield stress (which is taken to mean the effective section of the web in accordance with Clause 4.4(2)). Based on such effective sections, calculations have been undertaken of the contribution to the moment resistance of S235 grade webs with either S235 or S460 flanges. With web depth/thickness ratios of up to 200, it was shown that with S460 flanges the contribution would not be less than 75% of that with S235 flanges, implying that no more than 25% of the web moment would be assumed to be shed to the flanges which was considered to be acceptable.

NA.2.4 Basis [BS EN 1993-1-5:2006, 5.1(2)]

The buckling shear resistance in the Eurocode is given by:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}}$$

The recommended value of η given in the Eurocode is 1.2 for steel grades up to S460. This value was recommended on the basis that tests have shown that strain hardening allows a higher resistance to be mobilised without excessive deformation occurring - see background paper by Johansson, Maquoi and Sedlacek[4]. Both 3-1-1/6.2.6 and 3-1-5/5 reference the factor, η, to take this into account. This factor is defined in 3-1-5/5.1(2) but its numerical value is subject to national choice. For grades above S460, η =1.0 is recommended since strain hardening is less significant with higher steel grades.

The UK National Annex gives a more cautious approach because there is no experience of using higher shear stresses in design than $f_y / \sqrt{3}$ in the UK (the limiting value in BS5400 Part 3) and because, although typical ratios f_u/f_y for steels specified to Euronorms suggest that η = 1.2 is acceptable, the ductility requirement in the UK National Annex to EN 1993-1-1 only requires f_u/f_y to be greater than 1.1. Since it was considered that there was insufficient testing to support the higher recommended value of η in EN 1993-1-5, the UK opted to set the value at 1.0. Reference 3 provides greater discussion on this subject.

NA.2.5 Reduction Factor k_F for Effective Length for Resistance [6.4(2)]

For webs with longitudinal stiffeners, the National Annex allows the recommended value of k_F provided to be used for type (a) patch loading in 3-1-5/Figure 6.1:

$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 + \left[5.44\frac{b_1}{a} - 0.21\right]\sqrt{\gamma_s}$$

where $\gamma_s = 10.9\left(\frac{I_{sl,1}}{h_w t_w{}^3}\right) \le 13\left[\frac{a}{h_w}\right]^3 + 210\left[0.3 - \frac{b_1}{a}\right]$ with $I_{sl,1}$ equal to the second moment of area of the effective section (comprising the stiffener outstand and an attached width of web of up to $15\varepsilon t_w$ on each side) of the longitudinal stiffener nearest the loaded flange. This equation is based on the resistance of an unstiffened panel for the type (a) loading configuration with some additional resistance arising from the restraint to the web panel offered by the longitudinal stiffeners. The equation is only valid for $0.05 \le b_1 / a \le 0.3$ and $b_1 / h_w \le 0.3$ where b_1 is the depth of the sub-panel adjacent to the loaded flange. Beyond this limit, the formula for an unstiffened web should be used. Reference 5 provides greater discussion on this subject.

The formula is only valid for type (a) loading. For situations other than type (a) loading, the UK National Annex requires the appropriate formulae for an unstiffened web to conservatively be used or a buckling analysis undertaken. The National Annex does not specify what type of analysis this might be. It is possible to determine the elastic critical patch load from a finite element analysis for use in slenderness calculation. If this is done, the plate boundaries must be modelled with hinged edges in order to be compatible with the analysis behind the derivation of the reduction factor curve in 3-1-5/6.4(1). Alternatively, non-linear analysis with imperfections could be used to determine the resistance to patch loading and other accompanying actions directly.

NA.2.6 Flange Induced Buckling [8(2)]

The EN 1993-1-5 clause merely provides a limit to web slenderness related to the out-of plane curvature of a compression flange and the flange yield stress. It does not make an allowance for unintentional curvature of the flange relating directly to fabrication tolerances (although reference 3 illustrates how some allowance has been made for this in 3-1-5/(8.2)) and nor does it include an interaction of the effects of radial force on the web with those from bending and shear in the main girder. The NA provides rules to explicitly account for these two additional aspects.

Calibration studies have shown that with practical ranges of geometric and material parameters for beams of constant depth with differing intended curvatures and with k=0.55, the limit to web slenderness would provide a factor of about 2 against Euler buckling of an unstiffened web. However if a web is stiffened transversely, a factor of 2 against the critical buckling stress would be obtained at greater slenderness.

When the flange is intentionally curved, it is necessary to consider an interaction of the transverse force on the web with the main girder bending moment and shear. To do this, the NA introduces a check based on the patch load rules of 3-1-5/6 and their interaction with other effects in 3-1-5/7.2. In the absence of full calibration of this approach, a partial factor of 2.0 has been introduced to the patch load resistance for consistency with the above noted factor against elastic buckling of the web under transverse force alone. This approach is likely to be conservative and may be refined in later revisions of the NA. A more rigorous buckling analysis can of course be used instead of these provisions.

Reference 3 provides more background to these additional NA requirements.

NA.2.7 Stiffeners and Detailing [9.1(1)]

3-1-5/9.3.3(3) requires transverse stiffeners to be checked for the difference between the applied shear and the elastic critical shear force of the web panel. This is not strictly compatible with the rotated stress theory used in the shear design in 3-1-5/5 which does not require the stiffeners to carry any load other than the part of the tension field anchored by the flanges, corresponding to the term $V_{bf,Rd}$. In the absence of a stiff flange to contribute to $V_{bf,Rd}$, the stiffeners simply contribute to elevating the elastic critical shear stress of the web according to this theory.

Despite this theory, stiffeners do in reality develop stresses from compatibility of deflections, because their presence keeps the web flat at the stiffener locations, which changes the state of stress in the web. As a result, 3-1-5/9.3.3(3) does require a stiffener to carry a force equal to the shear force in excess of that required to cause elastic critical buckling. This leads to the stiffener design force being:

$$P_{Ed} = V_{Ed} - \frac{h_w t \tau_{cr}}{\gamma_{M1}}$$

This equation was not universally agreed by the EN 1993-1-5 Project Team at the drafting stage. It was generally believed to be overly conservative.

BS5400 Part 3 followed a similar approach which was compared against tests by H R Evans and K H Tang[6] for beams without longitudinal stiffeners and found to be slightly conservative but "not unreasonably so". Notably however, no stiffeners actually failed, even in the test designed to produce stiffener failure. The BS 5400 approach however also allows for the possibility of elastic critical buckling occurring at a shear stress less than τ_{cr} when direct stresses from bending and axial force are present in the web panels. Such considerations lead to significant discrepancy with EN 1993-1-5 for beams with unequal flanges, and hence significant average web compression, (making EN 1993-1-5 much less conservative) and hence the UK National Annex sought to modify the force requirement in the Eurocode to be in line with that in BS 5400 Part 3. This produces the requirement in the National Annex that:

When applying 9.3.3(3), the design force in the NOTE should not be used. Instead, intermediate rigid stiffeners should be designed for an axial force, P_{Ed}, acting in the mid-plane of the web in addition to any externally applied loads, where:

$$P_{Ed} = V_{Ed} - h_w t \times 0.8\tau_{cr}\sqrt{1 - \frac{\sigma_{x,Ed}}{0.8\sigma_{cr,x}}}\ \textit{for}\ a \geq h_w$$

$$P_{Ed} = \left[V_{Ed} - h_w t \times 0.8\tau_{cr}\sqrt{1 - \frac{\sigma_{x,Ed}}{0.8\sigma_{cr,x}}}\right]\frac{a}{h_w}\ \textit{for}\ a < h_w$$

where:

τ_{cr} is the elastic critical shear buckling stress determined in accordance with 5.3;

$\sigma_{cr,x}$ is the elastic critical plate buckling stress for direct stress determined in accordance with 4.4 or 4.5 for sub-panel buckling and overall stiffened plate buckling respectively;

$\sigma_{x,Ed}$ is the greatest longitudinal compressive stress in the web panel under consideration; $\sigma_{x,Ed}$ is taken as positive when compressive and should not be taken greater than $0.8\sigma_{cr,x}$.

When applying 9.2.1, $\sigma_{cr,c}$ and $\sigma_{cr,p}$ should generally be calculated for a panel length of $a_1 + a_2$ with w_0 taken as the lesser of b/300, a_1/300 and a_2/300. However, to safeguard against a mode of buckling with adjacent transverse stiffeners bowing in alternate directions, 9.2.1 should also be applied with $\sigma_{cr,c}$ and $\sigma_{cr,p}$ calculated for a panel length of 0.5 ($a_1 + a_2$). In this case, w_0 should be taken as the lesser of b/150, a_1/150 and a_2/150.

A non-linear finite element parametric study of over forty different cases of varying beam geometries, moment-shear ratios and axial force has been carried out in reference 7. In all cases, the EN 1993-1-5 rules (prior to UK National Annex amendments) were shown to be safe. Further, in every case tested, the stiffness requirement of 3-1-5/9.3.3(3) on its own would have sufficed as a design criterion. The behaviour observed was very much as predicted by the rotated stress field theory of Höglund. Up until a shear stress of around the elastic critical value, a linear distribution of bending stress occurred across the depth of the cross section. Beyond this shear stress, a membrane tension developed which modified the distribution of direct stress in the girder. This gave rise to a net tension in the web which was balanced by opposing compressive forces in the flanges, adding to the flexural compressive stress in one flange and reducing the flexural tensile stress in the other. This behaviour gives an increase in compressive flange force beyond that predicted solely from a cross section bending analysis, but not from that predicted by the EN 1993-1-5 shear-moment interaction in 3-1-5/7. For cases with strong flanges, some additional tension field was anchored by the flanges and the force transferred to the stiffeners. The conclusion was that the rules of EN 1993-1-5 were themselves somewhat conservative and needed to be relaxed, not tightened up. A proposal for a reduced design force is currently being considered for the next revision of EN 1993-1-5 so this UK National Annex provision may need to be reconsidered at the next update.

NA.2.8 Minimum Requirements for Transverse Stiffeners [9.2.1(9)]

The basis of the limiting stiffener proportions given in BS5400:Part3 was that the critical torsional buckling stress should exceed 2.25 times the yield stress[8]. This factor was derived by non-linear analysis as that required to ensure that the strength of a flat stiffener without rotational restraint with imperfections equal to one and a half times the tolerance would not fall by more than 2.5% from that for a perfectly flat plate. The same safety factor was subsequently adopted when rotational restraint from the attached plate was included and for other types of stiffener. This factor roughly corresponds to a slenderness plateau length of 0.7 on the reduction factor – slenderness curves of EN 1993-1-5 and is appropriate for plate behaviour, as is appropriate for flat stiffeners with limited warping resistance.

It has been shown that the criterion in the EN would not be met for a range of tee and angle stiffeners which just comply with the limits in BS5400. Analysis of the critical buckling stress for the same range of sections has shown that without parent plate restraint the values of θ in 3-1-5/(9.4) would have to be as low as 0.5 to demonstrate adequacy because the BS allows for plate restraint . The EN ignores such restraint to avoid consideration of the loss of plate stiffness with increasing plate stress which was accounted for in the derivation of the BS rules; a pan-European approach could not be agreed during drafting. Typically a stiffener complying with 3-1-5/(9.3) would provide θ >2 without plate restraint. Accepting that in the worst scenario that the plate restraint could be low it was recommended that θ = 2 in the NA, the warping stiffness being calculated with the centre of rotation being taken as at the plate surface. This produces a plateau length of around 0.7 as noted above which may seem rather long compared with a plateau length of 0.2 for column like behaviour more analogous to warping restraint. The longer plateau length has been justified in the NA by this absence of consideration of the parent plate restraint. The resulting limitations are still more severe than those in BS 5400 Part 3.

NA.2.9 Reduced Stress Method [10(1)]

No limits of application were placed on the use of clause 10 in the UK National Annex for three principal reasons:

(i) There are geometrical restrictions to the effective section method of section 4 which means that without the availability of section 10 no method would be available in some circumstances.

(ii) Other parts of EN 1993-2 refer to this section for derivation of a reduced limiting stress, σ_{limit}, to be used in checks under bending and axial load.

(iii) The method was considered to generally be conservative compared to that in section 4.

Picking up on the last point, the method is not always conservative. Since, unlike the method of section 4, no account is taken of the beneficial shedding of load from overstressed panels. It can therefore be conservative by comparison, although it is not always conservative where hand calculations are used[3]. Additionally, since shear stresses and transverse direct stresses are considered directly in this method, no further interaction between these different effects needs to be considered. This is another potential area of conservatism as shear stresses and transverse stresses, whatever their magnitude, have an immediate effect on the resistance to direct stresses, whereas this is not the case when the interaction-based approach with effective sections is used.

Since drafting the UK National Annex, it has further been noted that the method of section 10 can be significantly unconservative for cases of biaxial compression. ECCS Working Group 8.3 are looking at correcting the formulation of the equations to remove this unconservatism.

If the whole member is prone to overall buckling instability, such as flexural or lateral torsional buckling, these effects must either be calculated by second order analysis and the additional stresses included when checking panels to section 10 or by using a limiting stress σ_{limit} when performing the buckling checks to EN 1993-2 clause 6.3. For flexural buckling, σ_{limit} can be calculated based on the lowest compressive value of axial stress $\sigma_{x,Ed}$, acting on its own, required to cause buckling failure in the weakest sub-panel or an entire panel, according to the verification formula in 3-1-5/10. This value of σ_{limit} is then used to replace f_y in all parts of the buckling check calculation. It is conservative, particularly when the critical panel used to determine σ_{limit} is not at the extreme compression fibre of the section where the greatest direct stress increase during buckling occurs. For lateral torsional buckling, σ_{limit} can be determined as the bending stress at the extreme compression fibre needed to cause buckling in the weakest panel. This would be very conservative if σ_{limit} was determined from buckling of a web panel which was not at the extreme fibre for the reason above; the web panel stress would not increase much during buckling.

NA.2.10 Reduced Stress Method [10(5)]

A problem arises with the use of equations (10.4) or (10.5) in panels where the stress is tensile throughout or where there is stress reversal such that the compressive stress at one fibre is less in magnitude than the tensile stress at the opposite fibre. In the latter case, the greater tensile stress potentially ends up being magnified by the reduction factor determined using the critical stress for the compression zone if $\alpha_{ult,k}$ and the check in (10.4) or (10.5) are evaluated using a tensile value of $\sigma_{x,Ed}$. This would be very conservative. In response to this problem, NOTE 2 of 3-1-5/10(5)b) recommends that the check is only applied to the compressive part of the plate and the UK National Annex adopts this recommendation.

It is rational to apply the method to the compressive parts. For direct stress alone, but with stress reversal as shown in Figure 3, the slenderness according to 3-1-5/4.4(2) is given by:

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}}$$

and since $\sigma_{cr} = \alpha_{cr}\sigma_{comp}$

$$\bar{\lambda}_p = \sqrt{\frac{f_y / \sigma_{comp}}{\alpha_{cr}}} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} \text{ with } \alpha_{ult,k} = f_y / \sigma_{comp}$$

Clearly in this case the slenderness is based on the compression fibre, even though the tensile stress is greater in magnitude.

Despite the recommendation of NOTE 2 of 3-1-5/10(5)b) and its adoption by the UK NA, a check on the tensile zone should still however be made as the tensile stress in conjunction with the shear stress may cause yielding before yielding due to buckling occurs in the compression zone. There are several options for conducting such a check and these are illustrated in reference 3. If $\sigma_{x,Ed}$ is tensile throughout the panel being checked, the reduction factor ρ_x could be taken as 1.0, although this is not explicitly covered by EN 1993-1-5.

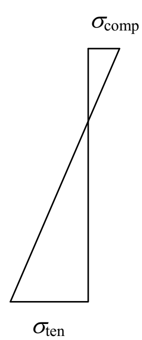

Figure 3 – Bending stress distribution across a plate where tensile stress exceeds the compressive stress

NA.2.11 Use of FEM [C.2(1)]

The UK NA states that the conditions for the use of FEM analysis in design should be specified for the particular project. The detail of the idealisation and analysis method used was considered to be best covered in an Approval in Principal document for the particular project.

NA.2.12 Use of Imperfections [C.5(2)]

EN 1993-1-5 allows two methods of calculation for imperfections; equivalent geometric imperfections covering both fabrication tolerances and residual stresses, or explicit consideration of these two components. Where allowance is made directly for fabrication tolerances and residual stresses, EN 1993-1-5 recommends only 80% of the fabrication tolerance is applied where this is applied with a distribution equal to that for the critical plate buckling mode. This reduction is justified by the low probability of having both the worst distribution of imperfection and the worst allowable magnitude and was therefore adopted in the UK National Annex.

EN 1993-1-5 does not elaborate on what is meant by the *critical buckling modes*. This is likely to be read as the critical *elastic* buckling modes but the lowest ultimate load factor is usually obtained by using a scaled version of the ultimate collapse mode as an imperfection; this is therefore iterative.

NA.2.13 Limit State Criteria [C.8(1)]

Figure 4 below shows a typical tensile stress-strain curve for grade S235 structural steel provided by the Corus Group on which have been superimposed the models c) and d) from Figure C.2 in EN 1993-1-5 clause C.6. The models are based on a yield stress of 235 MPa, not on the upper yield stress in the test. It is apparent that both models provide a good fit up to a strain of 5% but become non-conservative at higher values. Figure 5 shows a similar comparisons for grade S355 steel which with a higher initial strain hardening effect shows both models to be conservative at 5% strain but model d) becoming non- conservative at higher strains.

The limit to principal strain for regions subject to tensile stresses was therefore taken as 5% in the UK National Annex, regardless of stress-strain model used.

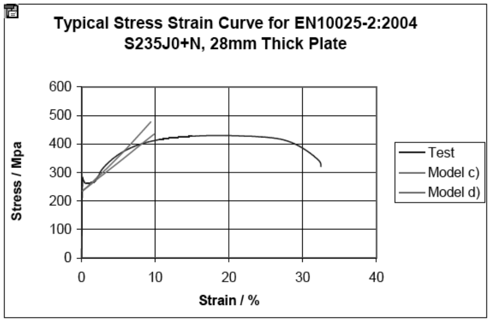

Figure 4. Stress-strain behaviour for S235 steel

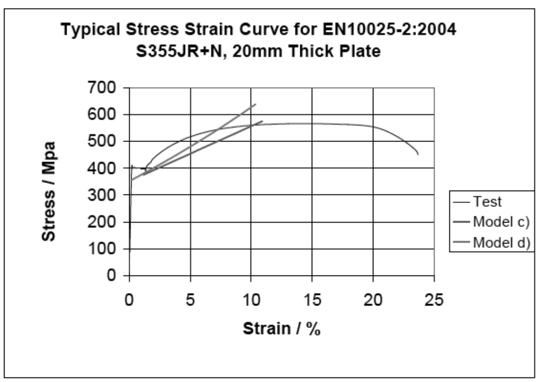

Figure 5. Stress-strain behaviour for S355 steel

NA.2.14 Partial Factors [C.9(3)]

The UK National Annex adopts the recommended value of material factors since these were the basis for the target reliability in Eurocode 3 and were also the basis of the calibration of Eurocode 3 with previous UK practice.

NA.2.15 Shear Resistance [D.2.2(2)]

The UK has little prior experience of the use of corrugated webs so the recommendations of the EN 1993-1-5 project term were adopted.

Acknowledgements

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EN 1993-2: PD 6695-2: RECOMMENDATIONS FOR THE DESIGN OF STEEL BRIDGES

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Abstract
The paper presents the background to the development of the provisions of *PD 6695-2:2008 Recommendations for the design of bridges to BS EN 1993*. That Published Document was prepared with the objectives of providing information on topics not covered by BS EN 1993-2 and offering guidance where it was considered further explanation of the Eurocode provisions was desirable for their correct and consistent application. It explains that the main sources of this material were BS 5400-3 and *Designers' Guide to EN 1993-2, Eurocode 3: Design of steel structures. Part 2: Steel bridges*.

Introduction
The objective of this paper is to give the background to the development of the provisions of *PD 6695-2:2008 Recommendations for the design of bridges to BS EN 1993*. The Published Document was prepared by B525/10 Working Group 3 and is referred to in the National Annex to BS EN 1993-2 as a source of NCCI. The PD was written with two primary objectives in mind:

(i) Provision of information on topics not covered by EN 1993.
(ii) Provision of guidance where it was considered further explanation of the Eurocode provisions was desirable for their correct and consistent application.

The first objective was the subject of debate during drafting because the principle-based approach used in the Eurocodes, together with the wide range of analysis techniques permitted, combine to ensure that it is usually possible to design all elements of a bridge utilising the Eurocode methodologies without further information, if a sufficiently powerful analysis model is used. However, the drafters of the PD considered it undesirable to *require* an increase in the level of complexity of analysis over and above that used in previous practice, although the flexibility to *permit* such analysis was considered to be beneficial. The material included to meet (i) is therefore usually in the form of design rules that can be applied by hand methods of calculation with a similar level of complexity as required by previous practice to BS5400. Much of the PD draws on recommendations in BS 5400-3, where they are non contradictory, as they are perceived by many to be more user friendly than Eurocodes. Clarification of Eurocode provisions also draws on guidance in Reference 2.

The paper deals with each principal PD clause in turn and provides explanation for the recommendations given. References to clauses in EN 1993-2 have been abbreviated below. For example, 3-2/6.3.4.2(5) is a reference to clause 6.3.4.2(5) of EN 1993-2. The PD clause numbers are given in parenthesis in each heading. It should be noted that the clause numbering in the PD does not follow that in EN 1993-2.

Global analysis (4)

Joint modelling (4.2)

The statement that semi-rigid (or semi-continuous joints as termed in EN 1993-1-8) should not be used for bridge structures has been included because such details are difficult to assess for fatigue performance, are likely to have relatively short life and are not included in the various detail categories of EN 1993-1-9.

Truss connections (4.3)

The recommendations given for the considerations of truss connections in global analysis have been imported from the requirements in BS 5400-3[1]. They essentially assume that joint moments arising purely from joint stiffness and compatibility are shed at ULS, as was previous practice. This is in line with EN 1993-1-8 provisions.

Main beam splices and bracing member connections (cl 4.4)

According to 3-1-1/5.2.1(6), bolt slip needs to be included in analysis where it is significant but no specific guidance is given in EN 1993-2. The PD therefore provides guidance.

The PD requires that bolt slip should be taken into account at the connections of bracing systems because a sudden loss of stiffness arising from bolt slip leads to an increase in deflection of the main member and an increased force on the bracing member, which could lead to overall failure. Ideally, connections of bracing members should be designed as non-slip at ULS (Category C to EN 1993-1-8) to avoid the need to evaluate the effects of slip.

The PD permits bolt slip at main beam splices to be ignored in global analysis. It has been UK practice to design splice bolts to slip at ULS (Category B to EN 1993-1-8) without consideration of slip in global analysis. This is justifiable as, although slip could alter the moment distribution in the beam, splices are usually positioned near to points of contraflexure and therefore slip will not shed significant moment to either adjacent hog or sag zones. Also, the loading that gives maximum moment at the splice will not be fully coexistent with that for either the maximum hogging moment or maximum sagging moment in adjacent regions.

Corresponding recommendations are given in clause 19 of the PD for the design of the connections.

Imperfections (cl 4.5)

The PD introduces a recommendation that imperfections in common planarity of bearings should be allowed for in the analysis of torsional moments and reactions for torsionally stiff superstructures. Whilst such imperfections are permitted by the tolerances in EN1090-2, there is no explicit requirement for their consideration in design in EN 1993-2. It is however necessary because if the structure is stiff (for example a box girder with rigid diaphragms at supports) then an error in level of the bearings could induce significant torsion in the structure and an uneven load distribution on the bearings. This is primarily a problem at SLS but it could also trigger failure at ULS if there is inadequate ductility in the system (including bearings and supporting structures). The recommendations are consistent with the approach in BS 5400-3.

The PD also clarifies how to treat forces from imperfections in bracing systems comprising both torsional restraints and plan bracing. Where the restraint forces are to be transmitted to end supports by a system of plan bracing, the plan bracing system should be designed to resist the more onerous of the forces F_{Ed} from each restraint within a length equal to the half wavelength of buckling and the forces generated by an overall flange bow in each flange according to clause 5.3.3 of EN 1993-1-1. In the latter case, for a very stiff bracing system with zero first order transverse deflection, each flange applies a total force of $\frac{N_{Ed}}{62.5}\alpha_m$ uniformly distributed to the plan bracing, where α_m is the reduction factor for the number of interconnected beams in BS EN 1993-1-1 clause 5.3.3(1).

Non-dimensional Slenderness for Beams With Different Restraints (cl 5 to 8)

The basic definition of slenderness for lateral torsional buckling in 3-1-1/6.3.2.2 requires calculation of the elastic critical buckling moment, M_{cr}. Formulae for the elastic critical moment are not provided so the designer must find a way of determining this value. To do this, there are three basic means: refer to theoretical texts; determine a value directly from an elastic finite element model; use empirical formulae. It is not realistic to expect to find a suitable formula for M_{cr} from a text book; real bridge problems are too complex so the other methods must be considered.

It is becoming increasingly easy to calculate M_{cr} directly from a computer elastic critical buckling analysis, using a shell finite element model, and many engineers will find this the quickest and most accurate method. Some experience is required however to determine M_{cr} from the output as often the first buckling mode observed does not correspond to the required global buckling mode; there may be many local plate buckling modes for the web and flanges before the first global mode is found.

To avoid the need for computer analysis, it was decided to provide empirical rules in the PD that enable calculation of the slenderness for lateral torsional buckling without explicit calculation of M_{cr}. The rules have been adapted from those in BS 5400-3 and take advantage of the fact that for a Class 1 or 2 cross section

$$\bar{\lambda}_{LT} = \lambda_{LT}\sqrt{\frac{f_y}{\pi^2 E}}$$

and for a beam with Class 3 or 4 cross section

$$\bar{\lambda}_{\mathrm{LT}} = \lambda_{\mathrm{LT}} \sqrt{\frac{f_{\mathrm{y}}}{\pi^2 E} \frac{M_{\mathrm{el,Rk}}}{M_{\mathrm{pl,Rk}}}}$$

where λ_{LT} is the slenderness in BS 5400-3. Derivation of this equivalence is given in reference 2. Clauses 5 to 8 effectively import the rules from BS 5400-3 clause 9.7, covering situations without effective intermediate restraints, with effective intermediate restraints and with flexible intermediate restraints, transcribed into Eurocode terminology. These rules derive a value of slenderness from evaluation of 'effective length', which is a concept that is not needed when deriving M_{cr} from a theoretical analysis or from an elastic buckling analysis.

There is a semi-empirical approach to the determination of slenderness based on the use of the general method in 3-2/6.3.4.2. See further discussion below in relation to clause 9 of the PD. The general method cannot be used for cases where only torsional bracing is provided (e.g. paired beams during construction of the deck slab); in such cases, either the rules in PD clause 8 should be followed or M_{cr} determined from elastic critical buckling analysis.

Simplified Method for Verification of Lateral Buckling of Truss Chords and Flanges in Compression (9)

When an empirical approach is taken, rather than an FE analysis, the complexity of the PD formulae in clauses 5 to 8 mean it will often be preferable to use the simple compression chord model of 3-2/6.3.4.2. This is particularly applicable for U-frame bridges or completed steel and concrete composite bridges with a deck slab and with or without intermediate bracings in the span.

The simplified method in 3-2/6.3.4.2 is intended for use for beams where one flange is held in position laterally. The method is based on representing lateral torsional buckling (actually lateral distortional buckling, since one flange is assumed to be held in position) by lateral buckling of the compression flange. The method is primarily intended for U-frame type bridges but can be used for other flexible bracing systems as well. It can also be applied to lengths of girder compression flange between rigid restraints, as found in hogging zones in steel and concrete composite construction. For this method, the St Venant torsional stiffness of the beam is ignored. This simplification may be significantly conservative for shallow rolled steel sections but is generally not significant for most fabricated bridge girders.

The PD includes guidance on two aspects not covered by EN 1993-2. First, the expressions for U-frame stiffness in 3-2/Annex D do not contain a contribution from the flexibility of the joints between U-frame members. The PD offers suggested values of joint flexibility, in the absences of specific calculation. The values have been imported from BS 5400-3 (and are based on research by British Railways in the 1960s) and are acknowledged to be fairly conservative. Second, the expression for $N_{cr} = mN_{E}$ in 3-2/6.3.4.2(6) requires there to be rigid bracings at supports. In conventional U-frame decks, the end U-frames are normally not sufficiently stiff to be classed as rigid and hence an alternative expression for N_{cr} is required. The PD therefore provides a modified expression for m in this case. The expression has been derived from coefficients provided in BS 5400-3 (which were derived from consideration of beam on elastic foundation theory) which fulfilled the same purpose.

Restraints at Supports – Effects Due to Restraint of Main Beams (10)

Bracing providing torsional restraint at supports experience forces arising from imperfections such as lack of verticality of beams, lack of straightness in the flanges and bearing eccentricity. The PD provides simplified guidance on the design of torsional restraints at supports because EN 1993-1-1 only covers the subject indirectly through second order analysis of the main beams and bracing system with modelled imperfections. EN 1993-2 provides little guidance on this analysis although flange bow and lack of verticality imperfections are covered by 3-1-1/5.3.3 and would therefore allow a suitable analysis to be carried out. The rules provided derive from previous UK practice through BS 5400-3. Further commentary on this is given in reference 3.

Intermediate Restraints – Effects Due to Restraint of Main Beams (cl. 11)

Clause 11 of PD 6695-2 is intended as a clarification of the 3-2/6.3.4.2(5) requirements for design forces for intermediate bracings – see also the comments made in relation to PD clause 4.2 above. The expression for F_{Ed} in 3-2/6.3.4.2 does not cover torsional restraints without the presence of plan bracing, so the PD provides additional expressions to cover this situation. They are needed for the design of paired beams during construction. The formulae provided have been brought in from BS 5400-3, with modifications to notation to suit the Eurocode format.

The remainder of clause 11 treats the additional forces generated in U-frame bridges (including the flanges) by local loading on the cross girders. Loading on a transverse member will cause that transverse member to deflect and rotate at its connection to the vertical stiffener. The stiffener will therefore try to deflect inwards. If all cross girders are not loaded similarly, the tendency is to produce differential deflections at the tops of the stiffeners but this differential deflection is resisted by the flanges in transverse bending. A transverse force is therefore generated at the top of the stiffener and a moment M_{y} is produced in the flange. Simplified expressions for the force, F_{c}, and moment, M_{y}, are provided in the PD. They have been imported from BS 5400-3 and further background on their origins can be found in reference 3.

The method provided in the PD is simple to carry out but can give conservative results. Second order analysis carried out on a suitable 3-D model will produce more accurate results. Further guidance and background on this approach is given in reference 2.

Buckling Resistance of Plates With Out of Plane Loading (cl 12)

Where there is out of plane loading on a plate, as occurs for example in a longitudinally stiffened deck plate subjected to traffic loading, 3-2/6.5 can be used but it is far from comprehensive. The PD therefore provides two alternative methods of analysis; one in clause 12.2 based on the effective section method in 3-1-5/4 and one in clause 12.3 based on the reduced stress method of 3-1-5/10. In both cases, the rules have been set out so that as the transverse loading reduces to zero, the same result is obtained as would be derived from the use of the expressions in EN 1993-1-5 for in-plane loading alone. The expressions originate from reference 2.

The method in clause 12.2 is similar to that proposed in EN 1993-1-7 but ensures compatibility with EN 1993-1-5 which EN 1993-1-7 does not achieve. EN 1993-2 does not reference EN 1993-1-7.

The UK's decision on the use of 3-1-5/4 and 3-1-5/10 is discussed in the paper on the NA to EN 1993-1-5.

Generally the effective section method will be the more economic and thus the use of clause 12.2 will generally be the more economic when transverse load is present.

The reduced stress method provided in clause 12.3 has some limitations, notably that the minimum load amplifier α_{cr} (in 3-1-5/10) can legitimately be less than 1.0 due to the post-buckling reserve of plates, whereupon the amplification factor, $(1/(1-1/\alpha_{cr}))$ in PD clause 12.3 will become negative and the expressions become invalid. They will in any case become very conservative as α_{cr} approaches 1.0. This method is therefore not recommended unless the criteria for the use of 3-1-5/4 cannot be met.

Resistance of Members With Flanges Curved Out of Plane (cl. 13)

No guidance is given in EN 1993-1-1 on the design of beams with flanges continuously curved in elevation, mainly because it involves out of plane bending in plate panels, which is not explicitly covered by either EN 1993-2 or EN 1993-1-5. EN 1993-1-7 covers transverse loading (not curved beams specifically) but is not fully applicable to bridge members as discussed under clause 12 above.

Beams with vertical curvature develop out of plane bending moments in the flanges as shown in Figure 1. For I beams, this flange transverse bending is sometimes referred to as "flange curling". PD 6695-2 provides methods for calculating the stresses from curvature (similar to the rules in BS 5400-3) and then for combining them with other effects and verifying the section; reference is made back to section 12 for the latter purpose. For beams with longitudinal flange stiffeners, the main out of plane bending effect in the flanges is longitudinally between transverse restraints. Two options are provided in the PD to account for this. Either the effects from curvature can be represented by a transverse load or the curvature can be modelled as in increased imperfection in the stiffener which is included via the term Φ. The expressions originate from reference 2.

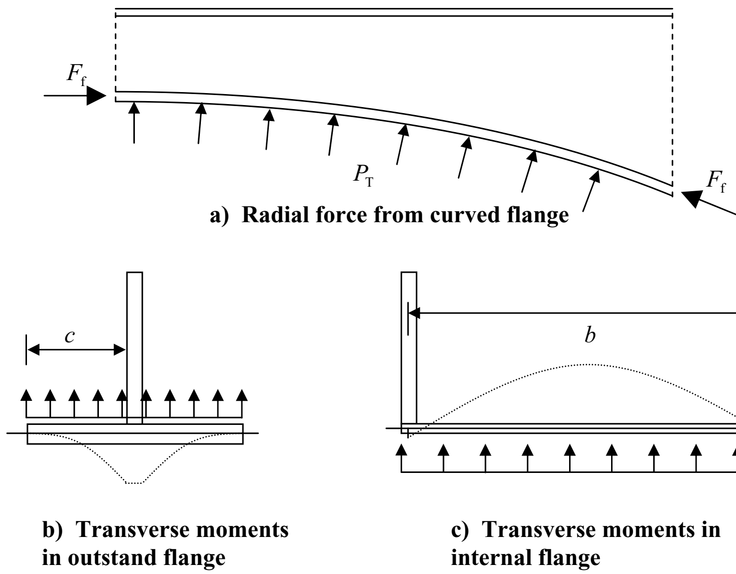

Figure 1. Forces and moments from flange curvature

Design of Flanges and Webs With Large Openings (14)

Clause 14 of PD 6695-2 has been added in order to ensure that openings in webs and flanges do not compromise post-buckling strength and fatigue performance and that designers consider secondary stresses that develop around openings. The principal recommendation here is that if the dimensional limits do not satisfy those in 3-1-5/2.3(1), the hole must be framed by stiffening and the surrounding area designed for secondary bending effects. This addresses the issue that large unstiffened edges of plates significantly reduce the post buckling strength and ductility of the section which could compromise the adequacy of many of the rules in EN 1993-1-5 which assume that such post-buckling strength is available.

Design of Intermediate Transverse Web Stiffeners (15)

Intermediate transverse web stiffeners are designed to 3-1-5/9. However, the clauses give only a description of the required performance of the stiffeners, which must satisfy a maximum deflection requirement and must not yield under all the design effects including second order effects. Also, EN 1993-1-5 does not provide a list of all the effects to consider acting on the stiffener, as was provided in BS5400-3.

To rectify the latter omission, the PD imports the list of effects listed for transverse stiffeners in BS 5400-3. It should be noted that this may not be an exhaustive list for all situations, which was the main reason for not including a list in EN 1993-1-5. For the strength and stiffness design, it had been intended to include formulae in the PD for calculating the deflections and stresses incorporating second order effects; paragraph a) in clause 15 of the PD refers to this intention. These formulae however were inadvertently omitted from the published version of the PD. They can be found in reference 2.

Design of Bearing Stiffeners (16)

The design of bearing stiffeners is also carried out according to 3-1-5/9. As for intermediate transverse stiffeners, the guidance provided in EN 1993-1-5 is much less comprehensive than the equivalent in BS 5400-3. As a result, the PD includes the following additional material, mostly imported from BS 5400-3:

(i) List of effects to consider in the design of stiffeners
(ii) Recommendations for bearing eccentricity values for different types of bearings
(iii)General detailing recommendations.

One important effect within (i) which was not imported from BS 5400-3 is the formula for web membrane force, N_H, which must be considered for the design of bearing stiffeners acting as rigid end posts. Reference 2 shows that the membrane force is given by:

$$N_H = h_w t_w \left(\frac{\tau^2}{\tau_{cr}/1.2} - \tau_{cr}/1.2 \right) \geq 0$$

where h_w and t_w are the height and thickness of the web panel respectively, τ_{cr} is the elastic critical shear stress for the web panel and τ is the shear stress. This formula has been derived from the assumptions underpinning the rotated stress theory used in shear design.

As with clause 15, for strength and stiffness design, it had been intended to include formulae for calculating the deflections and stresses incorporating second order effects but they also have inadvertently been omitted from the published version of the PD. They can be found in reference 2.

Connections – Design of Beam Splices (cl 17)

Splices in bridges are not explicitly covered by BS EN 1993-1-8, though general rules on the design of groups of bolts are provided. The additional guidance provided in the PD has, in the main, been imported from BS 5400-3 and has been included to bring a consistent approach to the design of splices. Of particular note are to the provisions of PD clause 17.5.4 which sets out how the forces and moments in each component part of a web splice should be derived from the overall moment, shear and axial force.

The specification of the Von Mises yield criterion in PD clause 17.4.2 as the means of verifying spliced plates and cover plates is also made for compatibility with existing practice to BS 5400-3. The PD recommedation is the same as the yield criterion is referred to in 3-1-1/6.2.1 as a general verification where no other rule is givenI.

Connections – Design of Gusset Plates (18)

Gusset plates are also not explicitly covered by BS EN 1993-1-8. Therefore guidance has been provided on the proportioning and verification of gusset plates to promote consistency. The detailing rules are based on design guidance previously provided in BS 5400-3 and the strength criterion is based on the Von Mises yield criterion, as discussed under clause 17 above.

Bolted Connections (19)

The guidance provided in this clause of PD 6695-2 is based on the same assumptions as outlined under clause 4.3 above.

Welded Connections (20)

Welded connections are extensively covered by EN 1993-1-8 but some additional non-contradictory recommendations were imported to the PD from BS 5400-3. The main recommendations are:

- that, for the fatigue design of welds connecting two parts in contact, all the force should be assumed to pass through the welds. This is because although at ULS the welds will deform as required to ensure that the two parts are in full contact (and thus the force can be transmitted in bearing), this condition cannot be assumed for fatigue because it cannot be assumed that the SLS loading has been exceeded, which would be necessary to bring the parts into full contact.
- that if weld leg length is specified, the throat used in calculations should not be assumed to exceed 0.71 times the leg length. If a bigger throat is required, it should be specified together with the leg length.

Cross Beams and Other Transverse Members in Flanges (21)

As for transverse stiffeners, cross beams and other transverse members in flanges should be designed to BS EN 1993-1-5. Guidance on loadings for determining design effects is provided in PD 6695-2. However, the means of calculating the deflections and stresses due to these loadings, together with second order effects, that is promised in clause 21 paragraph a) has been inadvertently omitted in the published version of the PD. Suitable formulae can be found in reference 2.

Shape Limitations for Stiffener Outstands (22)

Stiffener outstands may buckle locally in a torsional buckling mode transverse to the plane of the parent plate, possibly in combination with an overall global buckling of the stiffener out of the plane of the parent plate. Previous UK design practice (in BS 5400-3) provided guidance on shape limits for stiffener outstands which, when met, would be deemed to ensure torsional buckling could not occur. Such simple geometrical limits are not given in EN 1993-1-5. Instead, calculations are required to 3-1-5/9.2.1(8) and (9) to demonstrate that torsional buckling does not occur. Since some of these provisions require calculation of the critical stress, σ_{cr}, for torsional buckling, a formula for σ_{cr} has been provided. As this formula contains the warping constant, C_w, formulae for C_w for common stiffener types have also been provided.

For flat stiffeners only, it is possible to determine a simple shape limitation by following the EN 1993-1-5 rules, so this has been provided, namely that $\frac{h_s}{t_s}\sqrt{\frac{f_y}{235}} \leq 12.9$.

Box Girder Design (23, 24 and 25)

These clauses of PD 6695-2 introduce methods based on BS 5400-3 for evaluating torsional and distortional actions in box girders and for designing diaphragms. The recommendations are not strictly required because the general principles of EN 1993 can be applied to properly calculated effects that have been determined taking account of all structural behaviour.

It is likely that FE models using 2D shell elements will be used by designers to determine stresses for use in verifications. However, elastic FE analysis merges the longitudinal stresses due to axial force, bending (as modified by shear lag), torsional warping and distortional warping. When evolving a design, knowledge of the relative magnitudes of the separate components is useful because certain effects may be neglected at the ultimate limit state due to the occurrence of plasticity (effects such as torsional warping and much of the shear lag effects). The algebraic expressions provided in the PD allow these effects to be separated and are imported directly from BS 5400-3. They also provide a useful check on the output from FE analysis. A further alternative would be the use of non-linear FE analysis with modelled imperfections. This could automatically account for the effects of plasticity but is a much more involved and laborious process.

No attempt has been made to make the rules on diaphragms compatible with the Eurocode buckling curves and analysis methods; it was simply too great a task. Instead, the method has been described as “a self-contained procedure for hand methods”. This has been justified on the basis that diaphragms are not explicitly covered by EN 1993.

Web Breathing (26)

Clause 26 of PD 6695-2 was added as to clarify the requirements for web breathing in longitudinally stiffened sections provided in EN 1993-2. The clarification is simply that two verifications are required; one for the overall stiffened panel and one for the sub-panels between longitudinal stiffeners.

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THE UK NATIONAL ANNEX TO BS EN 1993-1-9:2005 AND PD 6695-1-9:2008

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Abstract

The paper gives a brief summary of the development of UK fatigue codes up to and including BS EN 1993-1-9 with particular reference to the Nationally Determined Parameters (NDPs) in its National Annex (NA) and the background to the choice of the UK parameters in BS EN 1993-1-9 and the supporting Non-Contradictory Complementary Information (NCCI) in PD 6695-1-9.

Background to Development of Bridge Fatigue Codes

Fatigue has been recognised as an important consideration in BS bridge design codes for nearly half a century. The widespread use of welding during and after World War II led to an urgent programme of fatigue testing in the UK in the 1950s. This resulted in the publication of the first comprehensive set of fatigue strength data for bridge design in an amendment to BS 153 Part 3B[1] in 1962. Fatigue testing of structural details continued during the 1960s and 1970s, both in the UK and aboard. The BS fatigue design rules for bridges were further refined as a result of the new data and those in BS 153 were replaced by BS 5400-10[2] in 1980, which also contained comprehensive rules for highway and railways fatigue loading. BS 5400-10 has been used as the basis for design of road and rail bridges in the UK and internationally for the last thirty years. It was probably the most comprehensive fatigue code for bridges in the world and in its early period was used as the basis for fatigue design in other structural applications.

At about the time of publication of BS 5400-10 work was initiated by the European Commission on the development of the Draft Eurocodes. In 1992 the draft steel Eurocode ENV 1993-1-1 was published to enable designers throughout Europe to test the new code on design projects for a two year trial period. BSI published the UK version as DD ENV 1993-1-1[3] together with a UK National Application Document which gave guidance on how to link with other British Standards for which European Standards did not yet exist.

Fatigue was covered in Chapter 9 of DD ENV 1993-1-1. This mainly consisted of a new set of stress range endurance curves based very much on the existing fatigue recommendations published by the European Convention for Structural Steelwork (ECCS). The latter also included fatigue curves for hollow section joints based on the CIDECT work. Chapter 9 was intended to cover all steel structures, not just bridges. It did not include any data for fatigue loading, which was to appear eight years later in DD ENV 1991-3:2000. The work of preparing ENV 1993-1-1 was undertaken by a Project Team (PT) funded by the European Commission. The experts had to cover all the technical subjects covered in ENV 1993-1-1, so the PT was very much reliant for fatigue input on the previous work of the ECCS.

The comments received from the CEN members, including BSI, on ENV1993-1-1 were considered by a new PT set up by CEN/TC 250 for drafting the final Eurocode 3 for steel. A decision was taken to remove a number of chapters from the ENV 1993-1-1, including the chapter on Fatigue which was transferred to a new part EN 1993-1-9, later to become BS EN 1993-1-9[4] in the UK.

During this period work by the UK bridge committee had been preparing revisions to BS 5400-10 and BS 5400-6 to link the fatigue requirements to the new workmanship standards. Revisions to both these British standards were published in 1999.

The basic fatigue curves in the draft EN 1993-1-9 had been updated from those in the ENV, and a new section with hot spot data added. The concept of design using 'damage tolerance' principles and two levels of 'consequences of failure' were included. The many comments received from CEN members on the ENV reflected a wide range of differing views around Europe. In order to accommodate this diversity and for acceptance by the CEN members it was decided to introduce a number of NDP options into the pr EN.

The UK view was that it contained some key deficiencies in comparison with BS 5400 Part 10 (see below). Their comments identifying the deficiencies were sent to the CEN committee who maintained that there were sufficient NDP options provided in the document to allow for the UK to include its requirements in the NA.

The final published version of EN 1993-1-9 contained eleven separate items with NDP options. Work started on the preparation of the UK's NA and associated NCCI material. Both were published in 2008, the latter by BSI as PD 6695-1-9[5].

The other significant development in 2008 was the publication of EN 1090-2, later to become BS EN 1090-2[6] in the UK, which is the construction specification on which the validity of the design rules depends. This was produced under the direction of CEN/TC135, and contained important clauses concerning fatigue, with particular reference to the influence of workmanship.

The significant changes made to EN 1993-1-9 using the NDP options in the NA should ensure that the resulting fatigue design is similar to that using BS 5400-10. It is hoped that the CEN committee considers the NA and PD 6695-1-9 in revising the Eurocode which is expected in about five years time.

Important Differences Between BS EN 1993-1-9 and BS 5400-10

The most important differences between the old and new standards, in so far as they affect bridges, are summarised below:

a) BS EN 1993-1-9 covers fatigue of all structural steelwork whereas BS 5400-10 was written specifically for bridges.
b) BS 5400-10 is a self-contained document for fatigue design, covering all the necessary data for resistance and traffic fatigue loading. BS EN 1993-1-9 only covers resistance, not loading which is covered in BS EN 1991.

c) BS 5400-10 is based on safe-life principles, ie an in-service inspection regime for detection of fatigue cracking is not a prerequisite. EN 1993-1-9 offers an alternative ‘damage tolerant’ option which does depend on in-service inspection.
d) The fatigue stress ranges in BS 5400-10 are based on principle stresses, with a cut off where maxima and minima are in directions differing by more that 45^o. EN 1993-1-9 uses ‘normal’ and ‘shear’ stress range whose damages are calculated and added together.
e) BS 5400-10 gives detailed information on what range parameters are to be used, where the initiation sites and crack propagation paths are for which the stresses are to be calculated, and when gross stress concentrations need to be included in the calculations. BS EN 1993-1-9 is not very clear on these matters.
f) BS EN 1993-1-9 recommends that the design strength is varied according to the ‘consequence of failure’. BS 5400-10 had no such recommendation, although it did provide information on probability of failure, which BS EN 1993-1-9 does not.
g) BS EN 1993-1-9 gives resistance data for certain hollow sections joints and certain ‘orthotropic deck’ details. Whilst these are not specifically listed in BS 5400-10 the principles could be used to obtain conservative values.
h) The stress-range-endurance curves have the following differences:
   - The range of strengths is slightly wider in BS EN 1993-1-9.
   - The curves increase in steps of about 12% in BS EN 1993-1-9 compared with typically between 13 and 20% in BS 5400-10.
   - The BS EN 1993-1-9 curves are designated by their fatigue strength in N/mm^2 at 2 million cycles which is the ‘detail category’, as opposed to the alphabetical ‘fatigue class’ in BS 5400-10.
   - The BS EN 1993-1-9 basic curves are all parallel on the log-log plot with an inverse exponent of 3, whereas the BS 5400-10 higher strength curves have a flatter slope with a higher inverse exponent.
   - Both sets of curves have a bend for variable amplitude purposes, whereby the exponent increases by 2. However the BS EN 1993-1-9 curves bend at 5 million cycles compared with 10 million in BS 5400-10.
   - The BS EN 1993-1-9 variable amplitude curves have a horizontal cut-off at 100 million cycles, whereas BS 5400-10 has no cut-off (ie the fatigue strength continues to decrease for higher endurances).
   - The BS EN 1993-1-9 design curves are notionally based on a 5% probability of failure. The BS 5400-10 design curves are based on a notionally 2.3% probability.
i) The BS EN 1993-1-9 detail category tables involving many more figures and text than was found necessary in BS 5400-10. Fields of parameters are laid out differently, for example dimensional information appears in columns 2, 3 and 4 and manufacturing information in both 3 and 4.
j) BS 5400-10 classification tables give warnings on the difficulty of assessing the necessary workmanship quality on the higher detail categories but there is none in EN 1993-1-9.
k) BS 5400-20 gives guidance to the designer on how to indicate the severity of fatigue stressing for the purposes of selecting the appropriate quality and inspection requirements to BS 5400-6 but no such guidance is available in EN 1993-1-9. There is however a very detailed informative annex in BS EN 1993-2:2006 which gives extensive dimensional and manufacturing information for a specific type of bridge

deck, The information does not appear to be dependent on the level of fatigue stressing, only static stressing.

l) Whilst both standards provide assessment methods based on limiting stress range and on damage summation, BS EN 1993-1-9 does not give tables of ready made values of these parameters for rapid fatigue checking of rail and road bridges. However λ values are given in BS EN 1993-2 which enable certain components to be checked by the first method.

m) Two methods of cycle counting are listed in BS EN 1993-1-9. The reservoir method is illustrated but the procedure is not explained. The rainflow method is referred to, but no information is given.

National Annex Decisions and NCCI in PD 6695-1-9

The differences between the BS EN 1993-1-9 and old fatigue codes described above, as they impact on bridges, have a major bearing on the decisions on the NPDs and the need for additional explanatory guidance. The bases of these decisions are explained below, with reference to the NA clauses in BS EN 1993-1-9.

NA.2.1.1 Material and execution tolerances

This clause made it clear that BS 5400-6[7] is an appropriate material and execution standard when designing to BS EN 1993-1-9, pending its final withdrawal following the publication of BS EN 1090-2. However, in order to select the relevant weld quality level, which is defined by the minimum class requirement in BS 5400-6 in terms of the alphabetically designated stress range endurance curves in BS 5400-10, the nearest equivalent numerically designated curves from BS EN 1993-1-9 are given for this purpose (see h) third bullet point).

PD 6695-1-9, Sections 2.1 and 2.2 give further explanation of the relationship between BS EN 1993-1-9 and BS 5400-6.

PD 6695-1-9 Section 2.3 gives provisional guidance on how the required quality level might be indicated on drawings when using the future BS EN 1090-2. This uses the system in ISO 10721[8] which is compatible with the BS EN 1993-1-9 detail category designation system. ISO 10721 uses the same 'FAT' symbol as in BS 5400-10 but with the numerical as opposed to alphabetical level designation (see Figure 1 in PD 6695-1-9).

Following eventual publication of BS EN1090-2 in 2008, the BSI bridge committee resolved to publish PD 6705-2[9] which gives guidance on the use of that standard for execution of bridges. PD 6705-2 recommends a similar method of determining production quality but uses the EN 1090-2 term 'service category' which is equivalent to 'fatigue class requirement' in principle, but extends Table 2 in PD 6695-1-9 to include different static stress levels. The designation 'F' is used in place of 'FAT' in the new context. As the co-existence period is now finished and BS 5400-6 and BS 5400-10 have now been withdrawn by BSI the NA and PD 6695-1-9 will be amended in due course to reflect this.

NA.2.1.2 Information on inspection requirements for fabrication

This draws attention to the limiting fatigue strength levels that can be assured by normal commercial inspection procedures (see j) above). In Tables 17a, b and c in the 1999 revision to BS 5400-10 many of the higher strength fatigue details from the 1980 version were bracketed and lower classes added without brackets. The bracketed values indicated that particularly high workmanship levels were required which would be beyond capability of normal inspection procedures to assess. Table NA.1 in the National Annex effectively provides the same information but in terms of the BS EN 1993-1-9 detail category designation, PD 6695-1-9 Section 2.4 discusses this matter in more detail.

NA.2.2.1 Sources of fatigue loading

The determination of specific fatigue load models is not likely to be of interest in the case of normal road and rail bridges, where the main source of fatigue loading is from traffic which is covered in BS EN 1991-2[10] (see b) above).

In cases where other sources of loading have to be derived from first principles, the guidance given in NA.2.2.1 and PD 6695-1-9 may be useful. The recommended values of γ_{Ff} in Table NA.2 are taken from BS EN 1999-1-3[11] (fatigue of aluminium structures). The recommended γ_{Mf} factors are to ensure that the low level of probability of exceeding the stress spectrum is achieved over the full life. (See NA.2.5.3 below).

NA.2.3 Determination of fatigue strength from tests

PD 6695-1 Section 4 gives general guidance on the main considerations for setting up specific fatigue tests. This is derived to some extent from the guidance in BS EN 1999-1-3, Annex C.

NA.2.4 Provisions for in-service inspection programmes

This is only applicable when a damage tolerant method of design is adopted (see c) above). This requires a very well defined regular in-service inspection programme to be maintained during the life of the structure together with a detailed method statement of repair of cracks and/or strengthening/repair of the member(s). The advice given in NA.2.4 has been based on that in BS EN 1999-1-3:2007, which has extensive guidance including fracture mechanics formulae used for estimating crack growth rates.

Items a) to h) in NA.2.4 of BS EN 1993-1-9 list the most important factors to be considered. It is essential that these are fully addressed and that the life time economic cost and practicality of using a damage tolerant approach is realistically quantified before any decision is made not to use the safe life method (see also NA.2.5.1 below).

NA.2.5.1 Assessment method

BS 5400-10 and BS 153 before it, has always been based on safe life principles (see ‘c)’ above). In order words the design method is intended to provide an acceptability low risk of fatigue failure during the whole design life. Routine inspection of bridges for fatigue damage is far more costly and time consuming than routine in inspection for other forms of damage or deterioration. It is normally only necessary when problems have been encountered on old bridges which may not have been designed or manufactured to adequate standards. The whole life cost of regular fatigue inspection programmes, including provision of access, restriction of use, possible paint removal, repair and/or strengthening, is potentially enormous.

There are very few exceptions to this practice in the UK. Military bridges, where minimum weight is an over riding requirements and which spend a lot of their life in storage (and hence are readily accessible and have replaceable components) are a case in point.

PD 6695-1-9 Sections 5.1 to 5.3 give further guidance on this subject.

NA.2.5.2 Classes of consequences

The question of varying the margin of safety according to the consequence of failure was discussed by the BS bridge committee at the time of drafting BS 5400-10, particularly in the context of redundancy of members. At the time a distinction was made between redundant and non-redundant members in the US structural fatigue design codes. Whilst it was accepted that better prior warning of catastrophic collapse of a structure might exist if fatigue failures occurred in redundant members, the overriding concern was to avoid all fatigue failures. Any evidence of fatigue failures would almost certainly give rise to substantial costs to the bridge owner, arising from loss of serviceability, repair/strengthening and the need to implement a regular fatigue damage monitoring programme.

NA.2.5.2 recommends that the basis of selecting a suitable 'consequence class' should be as recommended in EN 1990[12] Annex B and that the norm should be Consequence Class 2 (CC2). This is associated with Reliability Class 2 (RC2) which is what has been assumed in determining all UK γ_M and γ_F factors in BS EN 1993, BS EN 1990 and BS EN 1991 respectively.

PD 6695-1-9 Section 5.4 gives more background to the use of BS EN 1990 Annex B in deriving the NA.2.5.2 recommendations.

In the event that a particular case for changing the safety margin is agreed with the client, it has been recommended that this is done via k_{FI} on loading in accordance with BS EN 1990.

NA.2.5.3 Partial factor for fatigue strength

The recommended single value of γ_{MF} =1.1 is intended to provide a similar level of materials reliability to that used in BS 5400-10. PD 6695-1-9 Section 5.5 indicates the main differences in the stress range/endurance curves which have been taken into account in this process (see 'h)' above).

It should be noted that BS EN 1993-1-9 Table 3.1 effectively recommends increasing the design stress ranges by about 15% for damage tolerant designs, which is slightly more than one detail category. This is not acceptable in the UK; it is considered that such a design method should be based on a totally different procedure from that for safe life (see NA.2.4).

NA.2.6 Stress limitations for Class 4 sections

This guidance in BS EN 1993-2, 7.4 is considered appropriate to avoid web-breathing.
It should be noted that a similar formula to (7.7) in 7.4.(3) is given in BS EN 1999-1-3:2007, D.3(2), but is for general plate elements. This phenomenon is not confined only to web plates. Slender compression and tension flanges may also be susceptible.

NA.2.7 Use of nominal, modified nominal, and geometric stress ranges

PD 6695-1-9 Sections 6 and 7 give comprehensive guidance on this subject. This has made use of information provided in BS EN 1999-1-3:2007 which made use of material obtained from BS 5400-10 and BS 7608[13]. Guidance on the use of finite elements for fatigue analysis can be obtained from BS EN 1999-1-3:2007 Annex D.

NA.2.8 Design value of nominal stress range

Derivation of generalised λ data is a very specialised procedure, requiring extensive damage calculations based on a range of stress histories. Where λ values are not available from BS EN 1993-2 it is recommended in NA.2.8 that the specific member is assessed using the damage calculation method in Annex A (see also NA.2.11 below).

NA.2.9 Verification of fatigue strength category

The method of analysis of fatigue test data recommended in BS EN 1993-1-9 7.1(3) NOTE 1 is considered acceptable. For further guidance on fatigue testing see BS EN 1999-1-3 Annex C.

NA.2.10 Fatigue strength categories for details not covered by Tables 8.1 to 8.10 or Annex B

The fatigue strength categories in BS EN 1993-1-9 are normative and cannot be amended. They are considered to cover the most common details in steel bridges.

NA.2.11 Use of Annex A

PD 6695-1-9 Section 8 gives additional guidance on conducting the fatigue assessment using the damage calculation method, including the reservoir method as defined in BS 5400-10.

Conclusions

BS EN 1993-1-9 represents a very significant change from its predecessor BS 5400-10 as far as fatigue design of bridges is concerned, in terms of principles, scope, completeness, clarity and user friendliness.

The recommendations given in the National Annex and PD 6695-1-9 should ensure that bridges designed to the Eurocode will have a similar level of reliability and economy as those designed to BS 5400-10.

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THE UK NATIONAL ANNEX TO BS EN 1993-1-10:2005 AND PD 6695-1-10:2009

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Abstract

The Paper gives a brief summary of the development of the UK material toughness requirements up to and including BS EN 1993-1-10 with particular reference to the Nationally Determined Parameters (NDPs) in its National Annex (NA), the background to the choice of the UK parameters in BS EN 1993-1-10 and the supporting Non-Contradictory Complementary Information (NCCI) in PD 6695-1-10.

Background to Development of Material Toughness Requirements for Bridges

The brittle fractures on the Kings Bridge in Melbourne in 1962 brought the subjects of material toughness and weldability of welded steelwork into sharp focus around the world. This resulted in an amendment to BS 153, Part 1[1] in 1966 which restricted the thicknesses of tension members in terms of the very limited Charpy grades available at the time. It was also an important factor towards the publication of the first British Standard designed specifically for readily weldable structural steels, BS 4360[2] in 1968, which provided steels with improved chemistry and a wider range of Charpy properties. The BS 153 thickness range was amended to take account of these new steels.

Extensive work on fracture research for welded joints which started after World War II was continued into the 1960s and 1970s. As a result of improved understanding of the factors affecting fracture risk, including CTOD properties, residual and applied stresses, flaw size, temperature, rate of straining etc, further amendments to the bridge fracture rules were made, including those in the Merrisons' Box Girder Rules[3] in the early 1970s and eventually those in the new limit state code BS 5400-3[4] in 1982.

The BS 5400-3:1982 rules, which provided comprehensive thickness limits for all steel grades in terms of different service temperature, were made on the basis of type of detail, maximum applied tensile stress and the severity of gross stress concentrations. Almost identical rules were incorporated in BS 5950-1[5] for selection of Charpy requirements for buildings at about this time.

During the 1980s work progressed under the direction of the European Commission on the development of Eurocode 3. This resulted in the publication of DD ENV 1993-1-1[6] which contained a section with simplified Charpy requirements. DD ENV 1993-1-1 also contained informative Annex C which introduced parameters to take account of loading rate and consequences of failure which BS 5400 did not address. There was a two year trial period during which designers around Europe were intended to apply it. The resulting thickness limits for the various steel grades and Charpy qualities showed similar trends but some significant differences in values. DD ENV 1993-1-1, however, has never been used for designs in the UK.

In the 1990s a major review of the BS 5400-3 material toughness rules was carried out by TWI and UMIST for the Highways Agency, which resulted in a more comprehensive set of rules which were included in a major revision of BS 5400-3 in 2000. During this period a project team was set up by CEN/TC250/SC3 to consider all the comments of the members, (National Standards Bodies), on ENV 1993-1-1. At this point CEN/TC250/SC3 decided to remove the section of fracture requirements from Part 1-1 and publish it in a separate Part 1-10.

The technical development of Part 1-10 was carried out with the assistance of the materials group at Aachen University. This was taking place at the same time as the TWI/UMIST redrafting work for BS 5400-3. The results of the TWI/UMIST work were made available to the Aachen group. This ensured that the fundamental principles were harmonised between the two drafts as much as possible. There were however some major areas containing differences in assumptions which were not resolved. This included the role of residual stresses, the role of geometric stress concentrations and the size of undetected notches. The latter was somewhat uncertain as far as Part 1-10 was concerned, due to the absence of the execution standard EN 1090 which was in the early stage of preparation by CEN/TC135.

The new UK Charpy requirements for bridges in BS 5400-3:2000 contained thickness limits for each steel grade and Charpy sub-grade which were varied according to five levels of detail type, four levels of applied stress, the degree of gross stress concentration and the rate of loading. These were also used as the basis, with some minor simplifications, for the new Charpy requirements for buildings in BS 5950-1:2000.

EN 1993-1-10 introduced NDP options into that document which were sufficient to enable the thickness limits to be modified where a country considered it to be necessary.

EN 1993-1-10 contains only one set of thickness limits varying with steel grade and charpy sub-grade, 'reference' temperature and level of tensile stress were given. Adjustments to these thickness limits could be made by means of adjustments to the reference temperature T_{Ed} which allows for the minimum service temperature to be adjusted to cater for the effects of radiation loss ΔT_r, crack imperfection, member shape and dimensions, ΔT_σ, reliability differentiation ΔT_R, strain rate $\Delta T_{\dot{\varepsilon}}$ and degree of cold forming $\Delta T_{\dot{\varepsilon}_{cf}}$. This is a major difference from the presentation in BS 5400-3 where differentiation from the basic tables has always been done on the basis of a factor on the thickness limits. Adjustment using the temperature as a sliding scale can give similar results, but greater care needs to be exercised to ensure that the signs of the temperature adjustment are correct.

EN 1993-1-10 gives formulations for calculating the values of $\Delta T_{\dot{\varepsilon}}$ and $\Delta T_{\dot{\varepsilon}_{cf}}$, but not for ΔT_σ or ΔT_R, for which recommended values have been provided as default. In order to provide a degree of flexibility in interpretation of these undefined parameters National Annex provisions have been allowed to enable the basic thickness table values to be adjusted. The recommended method of making these adjustments is given in the National Annex to BS EN 1993-1-10 which refers to PD 6695-1-10 and is discussed below.

A new feature in EN 1993-1-10 was the addition of recommendations for selection of through thickness ductility requirements based on EN 10164[7]. This essentially gives protection against the risk of lamellar tearing in welded joints where weld shrinkage stresses can be generated in the through thickness direction (eg cruciform, tee and corner joints). The risk increases as the inclusion content (particularly sulphide inclusions) increases. The question was to whether some design guidance on specification of Z-quality steels should be included in BS 5400-3 has been discussed from time to time. However the view has been taken that the precautions to ensure that welded joints do not contain lamellar tears should be the responsibility of the fabricator. A survey carried out by TWI on behalf of the Highways Agency in the 1990s revealed very few reported problems of lamellar tearing at that time. One of the main reasons appeared to be that fabricators were aware that, provided steels were obtained from suppliers with modern mills and that the test certificates provided evidence of ‘clean’ material, they would normally expect to get a reasonable level of Z-quality without paying a premium for the necessary ultrasonic and through thickness testing to be done at the mill. If in doubt a fabricator could always request a limit on sulphur at the time of order.

The UK’s concern with the guidance of Table 3.2 was that, if followed literally, the extent of specification of Z-quality testing, even at the lowest level, would be invoked on a wide scale. This could have major implications for cost, availability of components (particularly for sections) and programme for steel bridge contracts. However there is a National Annex option with regard to the specifications of Z-quality and this is addressed in the NA to BS EN 1993-1-10 and PD 6695-1-10.

National Annex Decisions and NCCI in PD 6695-1-10

The differences between the new and old design codes with respect to selection of Charpy requirements and Z-quality properties described above, have a major bearing on the decisions on the NPDs and the need for additional guidance and design data. The bases of these decisions are explained below, with reference to the NA clauses in BS EN 1993-1-10.

NA.2.1.1 Safety element

The safety allowance ΔT_R has been used to enable a consistent safety margin to be obtained across the full range of different applications. The basis of the selection of the values of ΔT_R has been that it should give comparable margins of safety against brittle fracture to those given in BS 5400-3:2000.

This has been achieved by sub-dividing ΔT_R into five distinct safety elements, each dealing with specific parameters which can influence the overall safety margin. These have been given the following subscripts, ΔT_{RD}, ΔT_{Rg}, ΔT_{RT}, $\Delta T_{R\sigma}$ and ΔT_{Rs} which provide adjustments for detail type, gross stress concentrations, Charpy test temperature, applied stress level and strength grade respectively. The NA parameters ΔT_{RD}, ΔT_{Rg}, and $\Delta T_{R\sigma}$ are covered in BS 5400-3 by the parameters k_d, k_g and k_σ respectively (see clauses 6.5.3.2, 6.5.3.3 and 6.5.3.4 in BS 5400-3). The NA parameters ΔT_{RT} and ΔT_{Rs} are covered in BS 5400-3 by the formulae in Clause 6.5.4.

The ability to express these adjustments in terms of a temperature shift as opposed to a factor on thickness was possible by virtue of the fact that both BS 5400-3 and BS EN 1993-1-10 used the same basic formulation between fracture toughness (as measured in units of $Nmm^{-3/2}$) and temperature, ie the Wallin-Sanz correlation (see BS EN 1993-1-10 2.2(5) NOTE 2). This enabled a consistent transposition of the k thickness adjustments into equivalent temperature shifts to be made. For ease of use the resulting temperature shifts have been rounded to the nearest 10°C.

The values of ΔT_{RD}, ΔT_{Rs}, ΔT_{RT}, ΔT_{Rg} and $\Delta T_{R\sigma}$ are given in NA 2.1.1.2 to NA2.1.1.6 respectively. It should be noted that the only thickness values in Table 2.1 which are consequently not subject to adjustments are those for steel grades S355, σ_{Ed} = 0.75fy(t) and 'moderate' welded details. For example, at a minimum service temperature of -20°C the maximum permissible thickness for grade S355J2 is 50mm according to Table 2.1 in BS EN 1993-1-10 and Table 4 in PD 6695-1-10. However for plain material and 'very severe' welded details the maximum permitted thicknesses are 90 and 25mm respectively according to PD 6695-1-10, but unchanged from 50mm in BS EN 1993-1-10.

The reason why there is adjustment according to strength grade via ΔT_{Rs} and stress level $\Delta T_{R\sigma}$ is that significantly different assumptions about residual stress were made between those used for BS 5400-3 and those for BS EN 1993-1-10. In the former a value of yield stress was used (based on extensive knowledge of residual stresses in fabricated steelwork). In BS EN 1993-1-10 it is understood that a value of $100N/mm^2$ was assumed. If so, this would provide an explanation as to why the permissible thickness in BS 5400-3 is inversely proportional to the yield stress raised to the power of 1.4, whereas in BS EN 1993-1-10 it is raised to the power of 1.0. This also impacts on the allowable thicknesses for stress levels below 0.75fy(t).

The other difference concerns the adjustment for Charpy test temperature ΔT_{RT}. In BS 5400-3 there has always been a cut off service temperature whereby steels are not permitted to be used beyond a certain temperature drop from the Charpy test temperature. In BS 5400-3:2000 this was a drop of 20°C. In BS EN 1993-1-10 Table 2.1 JR material is permitted to be used at 70°C below the test temperature and many other grades at drops exceeding 20°C. In developing the UK NA, it was considered that this posed an unacceptable risk for which there was very little precedence in current brittle fracture rules. The ΔT_{RT} adjustment is designed to mitigate against this risk.

It was also considered that it would be useful for designers to have ready made look-up tables giving permitted thickness limits based on the formulations given in BS EN 1993-1-10. This would save the design industry a lot of calculation time and reduce the risk of error. PD 6695-1-10 was therefore produced for this purpose. Table 1 in PD 6695-1-10 gives the maximum thicknesses for a range of reference (note **not** service) temperatures and steel and Charpy grades. These contain none of the adjustments apart from ΔT_{Rs}. Table 4 gives maximum thickness limits for bridges for a service temperature of -20°C, (which covers the majority of UK sites), in terms of different detail types and stress levels. Equivalent Tables 2 and 3 for buildings have been given, the only difference being the slightly longer ramp down cut off limit which has been the practice for buildings in BS 5950-1.

NA.2.1.2 Charpy test and reference temperatures and applied stress
This clause effectively confirms that the options for adjusting thickness limits to allow for differences in Charpy test and reference temperatures is covered by the safety adjustments in Section NA2.1.1.

NA.2.1.3 Limitation of steel grade
This clause effectively confirms that the options for adjusting thickness limits according to stress level are covered by the safety adjustments in NA2.1.1.

Workmanship requirements
A final point that should be borne in mind is that BS EN 1993-1-10:2005 was published three years before BS EN 1090-2:2008. There was therefore some uncertainty about the level of workmanship which could be relied upon, particularly in respect of unspecified notches remaining in the work and the risk of their growth in service. PD 6705-2[7], which is in draft and scheduled to be published this year, addresses this particular issue and provides specific guidance on selection of appropriate choices and the supply of additional information to ensure that this matter is kept under sufficient control during execution that the Charpy selection criteria in BS EN 1993-1-10 will be valid.

NA.2.2 Quality class for through thickness properties
This option to adopt Class 2 in BS EN 1993-1-10. Table 3.1 effectively removes the requirement to specify Z-quality in accordance with Table 3.2. The designer instead is referred to PD 6695-1-10 Clause 3, which gives practical advice on what approach to take. This takes on board the comments above that the fabricator should be primarily responsible for taking precautions against lamellar tearing. However where ‘high risk’ design situations arise in welded tee, cruciform and corner joints it does recommend to the designer that Z-quality testing should be specified above certain thicknesses.

Conclusions
BS EN 1993-1-10 contains some similarities but also some significant differences as far as the selection of Charpy requirements for bridges are concerned, when compared to those in BS 5400. In view of the significant differences in the thickness limits between the UK NA Annex and the recommended values in EN 1993-1-10 for certain conditions, it is considered that opportunities should be taken in the future to reconcile the differences at the CEN level to ensure that structures designed to Eurocode will have a consistent level of safety across Europe.

EN 1993-1-10 is one of the few documents which have shown major divergence of views between the UK and other experts in Europe despite various attempts to reach a stronger consensus. It is clear that there is a need for improved understanding of the behaviour of structures susceptible to brittle fracture. Due consideration should be given to initiate further research at the European level to aid the development of consensus, particularly given the potentially dire consequences of sudden failure due to brittle fracture. Doing so should enable the current divergence in the NDPs used across Europe to be reduced.

The UK National Annex and its accompanying PD 6695-1-10 has brought design to Eurocodes in closer alignment with previous British practice, based on research carried out in the UK and abroad, than the recommended values of NDPs in EN1993-1-10.

Acknowledgements

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EN 1993 PRACTICE PAPER: BUCKLING ANALYSIS OF STEEL BRIDGES

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Abstract
Eurocode 3 presents definitions of slenderness in terms of critical forces or critical stresses to facilitate determination of slenderness from a computer elastic critical buckling analysis. This analysis will not always be required (there are often simpler provisions), but its availability allows more accurate slenderness determination than might otherwise be obtained by simple codified equations. Determination of slenderness can be avoided by carrying out second order analysis allowing for imperfections; this is a more time-consuming approach but will often given a more economic result, although not always depending on the imperfections used. The exceptions are discussed in the paper.

Introduction
Accounting for buckling is a key aspect of the design of steel structures. Eurocode 3 offers considerable flexibility to designers in the way that this can be done, and it is therefore important for designers to have an understanding of the fundamental concepts underpinning buckling behaviour, such as the implications of imperfections and slenderness, the effects of geometric and material non-linearity, and the possibility that buckling will occur at a global, member and/or local level.

Eurocode 3 presents definitions of slenderness in terms of critical forces or critical stresses to facilitate determination of slenderness from a computer elastic critical buckling analysis. This analysis will not always be required (there are often simpler provisions), but its availability allows more accurate slenderness determination than might otherwise be obtained by simple codified equations. Determination of slenderness can be avoided by carrying out second order analysis allowing for imperfections; this is a more time-consuming approach but will often given a more economic result, although not always depending on the imperfections used. The exceptions are discussed in the paper.

This paper provides some guidance on the use of both calculation methods including some areas where caution is required. A brief overview of buckling behaviour and analysis is also given. References to clauses in EN 1993 have been abbreviated below. For example, 3-1-5/3.3(1) is a reference to clause 3.3(1) of EN 1993-1-5.

General Overview of Buckling Behaviour and Analysis

In this section buckling behaviour and analysis requirements are considered in general terms; the specific requirements of Eurocode 3 are discussed in the following sections of the paper. It is easiest to first explore buckling behaviour in the context of a simple pin ended member under axial load, and such an approach is taken here. The observations are however, more generally relevant.

If a pin ended member with some initial (bow) imperfection is subjected to an increasing axial load, the member will tend to bow outwards until a point is reached when, with increasing lateral deflection, the load that can be sustained will reduce. The maximum axial load (referred to here as the buckling load) will be dependent upon the slenderness of the member, the initial bow and the material strength. The buckling load is usually reached when, or soon after, yield first occurs at an extreme fibre of the cross-section; yielding leads to a reduction in the (tangent) flexural modulus of the member and therefore the rate of change of lateral deflection with load increases.

The axial buckling load that can be sustained may be considerably lower than the (theoretical) maximum axial load that could be sustained by a perfectly straight member that remains in the elastic state. This theoretical maximum axial load is the *elastic critical force* (in Eurocode terminology) and for a pin ended member is the Euler buckling load, given by $\pi^2 EI / L_{cr}^2$, where L_{cr} is the member length.

Of course, there are other factors that affect the buckling load, for example:

(i) Residual (self-equilibrating) stresses in the member due to the way it has been manufactured can result in first yield, and therefore lateral instability, occurring at a lower axial load.
(ii) Local buckling of the plates that make up the member might occur, and whilst this local plate buckling might itself stabilise, it can result in a reduction in the effective stiffness of the member and therefore a reduction in the buckling load. This effect is usually very small.
(iii) If the member forms part of a larger structure, it is possible that some global buckling instability will occur, before the member reaches its buckling load.

The effects of geometric imperfections and residual stresses can be accounted for by incorporating appropriate geometric imperfections in the member buckling analysis as discussed in the section on *imperfections* below. It should be noted that the use of equivalent geometrical imperfections to represent residual stresses is usually more conservative than modelling the pattern of residual stress directly in the analysis.

There are essentially two methods that can be used to calculate the buckling load accounting for imperfections. The first is to use buckling curves that give a reduction factor that is applied to the resistance of the cross section (squash load); the reduction factor depends on the so-called 'non-dimensional slenderness' that expresses the relationship between elastic critical force and the squash load. In this way, the effect of buckling is taken into account through a reduction in the member resistance. This is the approach that has generally been used in past UK practice. The derivation of the buckling curves in EN 1993-1-1 is presented in reference 3.

The second approach is to model the imperfections in the member in a numerical analysis package that can take account of geometric non-linearity (*i.e.* the additional force effects arising from the lateral deflection of the member under axial load) and material non-linearity (i.e. yielding of the steel). There is generally no need to consider material non-linearity if the analysis is stopped when yield is first reached – the further increase in load is small. Both methods are discussed in this paper; if the imperfections are suitably chosen, both approaches can give identical results.

When the first of these two methods is used there are several ways in which the elastic critical force can be determined. In past UK practice, this was generally done through establishing an effective buckling length (typically using tables and graphs) and the same method can still effectively be used in designs to Eurocodes. However, with the increasing availability of software that can perform elastic critical buckling analysis, it is expected that the elastic critical force will increasingly be determined directly through numerical methods. There are a number of pitfalls when using software to perform elastic critical buckling analysis, and these are discussed later. There can also be considerable advantages.

It is, however, absolutely crucial that designers recognise that the results of elastic critical buckling analyses do not give the buckling load of the structure directly – they give ideal results (equivalent to the Euler buckling load) that must then be factored to account for imperfections.

There is one further aspect that merits comment, although it does tend to be more relevant to building than bridge design. This concerns the global response of the structure and the influence that it may have on member buckling. If a structure is globally sensitive to second order effects (*i.e.* if changes in its geometry under load give rise to increasing load effects), then it will be important that such second order effects are taken into account in determining the boundary conditions used for member buckling verifications, and in doing so, that account is taken of the effects of global imperfections. Finally, of course, it will be understood that consideration of imperfections is not only important in second order analysis, it is also important in evaluating the loads in bracing members *etc*.

A general overview of the approaches that can be taken to account for member and global buckling behaviour, the effects of imperfections and second order effects, is shown in Figure A.1 in Appendix A of this paper. In this figure, the term 'first order analysis' is used to refer to an analysis in which the deformed geometry is *not* taken into account, the term 'second order analysis' is used to refer to an analysis in which the deformed geometry *is* taken into account, and 'non-linear analysis' is used to refer to an analysis in which both geometric (*i.e.* second order) non-linearity and material non-linearity are modelled.

Elastic Critical Buckling Analysis

Use for buckling checks on members

Eurocode 3 presents the expressions for non-dimensional slenderness expressions in terms of critical forces, e.g. N_{cr}, M_{cr}, or, in the case of shear, in terms of critical stress, τ_{cr}. Some examples are shown below:

Buckling in compression: $\bar{\lambda} = \sqrt{\dfrac{Af_y}{N_{cr}}}$

Buckling in bending: $\bar{\lambda}_{LT} = \sqrt{\dfrac{Wf_y}{M_{cr}}}$

Buckling in shear: $\lambda_\tau = \sqrt{\dfrac{\tau_y}{\tau_{cr}}}$

It is therefore often beneficial to be able to evaluate these critical forces and stresses directly to determine the most accurate slenderness. The resistance to the mode of buckling being considered is then determined from equations for reduction factor for that particular mode, which is usually theoretically based and adjusted for test observations. The reduction factor curves for buckling of compression members in clause 6.3.1 of EN 1993-1-1 are a good example, having been derived from the Perry-Robertson theory using values of imperfections which provide good correlation with test results.

The critical stresses and forces can sometimes be obtained by hand calculation using mathematical expressions. This is quite easy for members in compression where the concept of effective length (L_{cr}) can be used to determine the critical buckling force N_{cr} as $\dfrac{\pi^2 EI}{L_{cr}^{\ 2}}$.

However, for bending the situation is different and it is difficult to determine an expression for M_{cr} for real bridge situations and hence determining the value directly from an elastic critical buckling analysis can be desirable and will often bring economic benefit.

Practical example of use

A good example of use is the buckling of paired beams during construction of the concrete deck slab. This may be a critical check as the girders will often be most susceptible to lateral torsional buckling (LTB) failure when the deck slab is being poured. Beams are normally braced in pairs with discrete torsional restraints, often in the form of X bracing or K bracing (as shown in Figure 1), but for shallower girders single horizontal channels connecting the beams at mid-height is an economic, but less rigid, alternative.

Figure 1. Pairs of braced beams awaiting deck slab construction

Paired girders with torsional bracing as above generally fail by rotation of the braced pair over a span length as shown in Figure 2. With widely spaced torsional bracing, buckling of the compression flange between bracing points is also possible. There are no formulae for the former situation given in EN 1993 so there are two possible approaches to determine a slenderness and hence the reduction factor for buckling:

(i) utilise the hand calculation method of PD 6695-2[1] to determine the slenderness directly;
(ii) determine M_{cr} by computer analysis for use in slenderness calculation.

The second method will produce the most economic design.

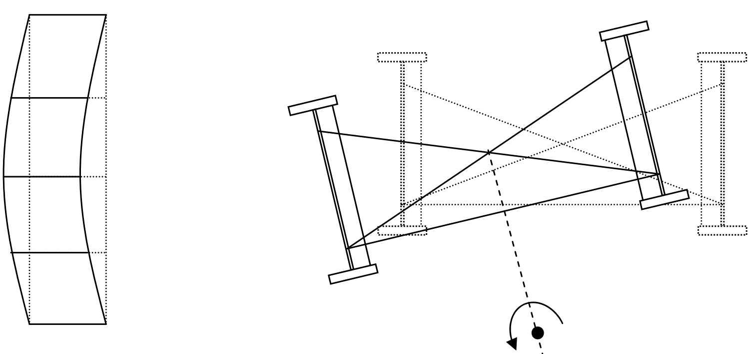

Figure 2. Buckling of paired beams prior to concrete hardening

An example composite bridge case is considered below. It is a simple single span bridge with two steel plate girders braced together by cross bracing. The dimensions are representative of typical UK construction, being based on a recently constructed bridge. A uniformly distributed vertical load was applied to both girders, representing ULS factored load from concreting of the span and steel self weight, and an elastic critical buckling analysis carried out. The lowest global mode of buckling, corresponding to the attainment of M_{cr}, is shown in Figure 3; the girder pair is seen to rotate together over the whole span. The second lowest global mode is shown in Figure 4 and corresponds to lateral buckling of the compression flange between braces. M_{cr} is obtained from the computer analysis as the largest initial first order bending moment multiplied by the load factor at buckling in the mode of interest.

Table 1 shows a comparison of the final bending resistances produced from method (i), (ii) and a full non-linear analysis (method iii), the latter being a very close approximation to the real bending resistance of the girders and is discussed more in below. The elastic critical bucking analysis method has a clear economic advantage over the hand calculation method. More detail on this particular example can be found in reference 2.

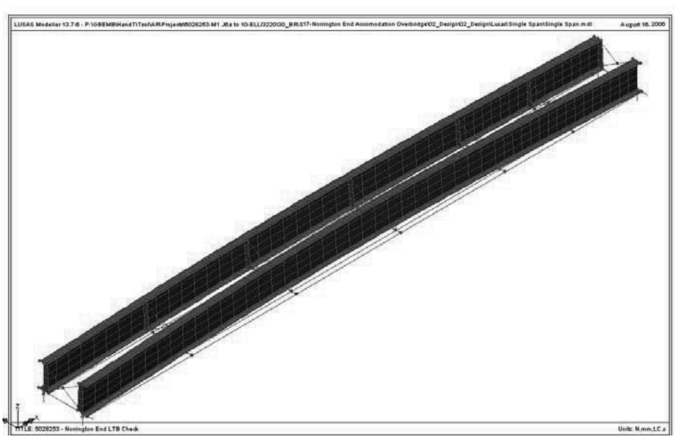

Figure 3. Lowest global mode of buckling for single span beams

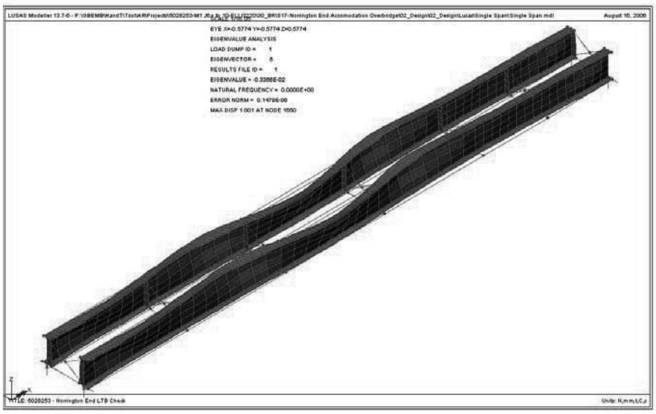

Figure 4. Second lowest global mode of buckling for single span beams

Calculation method		Bending resistance (kNm)
(i)	Hand calculation to PD 6695-2	5260
(ii)	EN 1993-1-1 clause 6.3.2 with M_{cr} determined from elastic buckling analysis	7470
(iii)	Non Linear FE (with strain hardening)	9591

Table 1. Comparison of resistances obtained by different methods for paired beams

Use for buckling checks on entire systems – BS EN 1993-1-1, 6.3.4

Clause 6.3.4 of EN 1993-1-1 is written as a general method for checking out of plane (lateral) buckling of members and frames when the axial force and bending moment *both* give rise to out of plane buckling of the element(s) i.e. the axial force or bending moment applied separately would lead to lateral buckling of the element(s). An example of this is given in the Designers' Guide to EN 1993-2[3], section 6.3.4.1, example 6.3-4. The slenderness for buckling is given by:

$$\bar{\lambda}_{op} = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr,op}}}$$

where $\alpha_{ult,k}$ is the load factor to apply to the factored ULS loads to cause cross section failure and $\alpha_{cr,op}$ is the load factor to apply to the same loads to give elastic critical buckling. In such cases, it is logical that the cross section resistance used in the slenderness calculation be based upon both the axial force and the bending moment together, because both cause lateral buckling of the system i.e.

$$\frac{1}{\alpha_{ult,k}} = \frac{N_{Ed}}{N_{Rk}} + \frac{M_{y,Ed}}{M_{y,Rk}}$$

An important caveat to this approach is that if there are significant *in-plane* second order effects (i.e. if the moment $M_{y,Ed}$ is significantly amplified by the presence of the axial force and in-plane deflections and imperfections) then these must be included in deriving $M_{y,Ed}$ and hence $\alpha_{ult,k}$. This is because since the moment $M_{y,Ed}$ leads to lateral buckling, its full value including second order effects must be used when checking lateral buckling.

The UK National Annex to EN 1993-1-1 limits the application of the rule to nominally straight members. This restriction was not intended by the Eurocode drafters; moments from initial curvature are included in the calculation of $M_{y,Ed}$ perfectly satisfactorily. Indeed, the example of application of the clause prepared by the Project Team[4] features a curved member.

The above format was not intended to be used to check other situations where the axial force and moment *do not both* promote out of plane buckling. The checking of arches is one such area, noting the limitations above in the UK NA regarding applicability only to straight members! The format could, with care, however be applied to arches. The application of the clause to the design of an arch is discussed in reference 5 where the method was shown to be acceptable.

Pitfalls in elastic critical buckling analysis

For those inexperienced in elastic critical buckling analysis, there are many pitfalls and some examples are given below:

(i) Not using the correct mode in calculations

The lowest global mode of buckling for the paired beams example above was shown in Figure 3. However, where shell elements have been used throughout, numerous local buckling modes such as that shown in Figure 5 will usually be found at much lower load factors. These typically correspond to buckling of the top of the web plate in compression or potentially to torsional buckling of the top flange and may be ignored for the purposes of determining M_{cr}; these buckling effects are considered in the effective section properties and flange outstand shape limits in codified approaches. These modes may occur at much lower load factors than the overall mode of buckling sought and their use in calculation would be very conservative. It is important that this is understood. Simpler models can sometimes be used to avoid determining modes that are of no interest e.g. the use of beam elements for flanges in plate girders to eliminate flange torsional buckling modes.

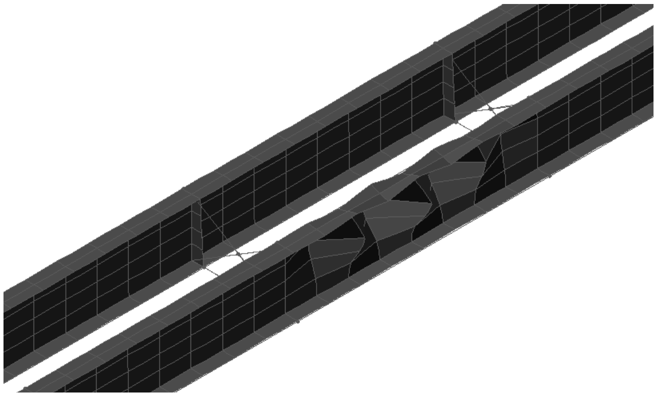

Figure 5. Typical local elastic buckling modes for beams

(ii) Not appreciating the limitations of software

Many, if not most, software programmes which can perform elastic critical buckling analysis do so based on the initial un-deformed geometry. Where a structure (e.g. an arch) or element flattens under load due to elastic shortening or abutment movements, the geometry changes and the compressive forces can increase as illustrated in the simplified system in Figure 6. Snap through buckling then becomes a possibility and this will be undetected by the software unless it can include the effects of geometric non-linearity.

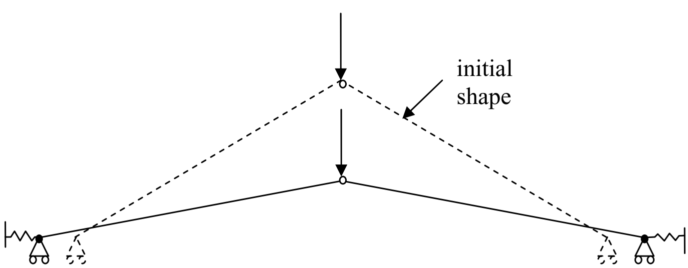

Figure 6. Flattening of arch (idealised as two pin-jointed members) due to abutment movement and elastic shortening

(iii) Not appreciating the limitations of code methods

The slenderness of arches can be determined by first obtaining N_{cr} from an elastic critical buckling analysis. This would be fine for the arch in Figure 7 with pin jointed hangers (not shown). The slenderness is determined, then the reduction factor is obtained and the arch strength is checked.

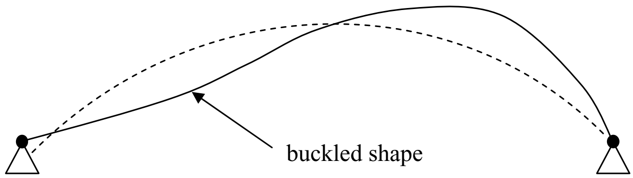

Figure 7. In-plane buckling of arch with pinned hangers

The same analysis can be used for the arch in Figure 8. However, this design has rigid connections at the ends of the hangers. The buckling deformations induce moments in the hangers but the analysis gives no information directly useful for checking the hangers. Consequently, arch buckling may be checked as above but the additional effects on the hangers are then missed if they are only designed for first order effects only. For this case, a second order analysis should be used to determine the hanger moments.

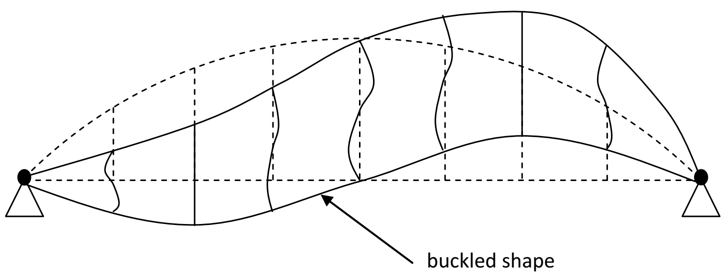

Figure 8. In-plane buckling of arch with rigid hanger connections

(iv) Not understanding the software output

The output of an eigenvalue buckling analysis is a series of buckling mode shapes and their corresponding load factors. Often, software also displays moments and forces with each mode. These are the internal effects associated with the mode shape when the peak displacement in that mode has been set to unity in some set of units. It is not therefore information that can be used directly in the design. Inexperienced engineers have however been seen to try and design against the moments produced.

Non-Linear Analysis

Imperfections

The modelling of imperfections is a key aspect to the non-linear analysis of structures. Imperfections comprise geometric imperfections and residual stresses. The term "geometric imperfection" is used to describe departures from the exact centreline setting out dimensions found on drawings which occur during fabrication and erection. This is inevitable as all construction work can only be executed within certain tolerances. Geometric imperfections include lack of verticality, lack of straightness, lack of fit and minor joint eccentricities. The behaviour of members under load is also affected by residual stresses within the members. Residual stresses can lead to yielding of regions of members at lower applied external load than predicted from stress analysis ignoring such effects, leading to a reduction in the member stiffness. The effects of these residual stresses can be modelled by additional equivalent geometric imperfections and these are given throughout EN 1993 for the overall design of members (e.g 3-1-1/5.3.1(2)**)** and for local buckling of plates (e.g. 3-1-5/C.5). Member imperfections can apply to overall structure geometries (global imperfection) or locally to members (local imperfection).

Imperfections must be included in global analysis unless they are included by use of the appropriate resistance formulae in clause 6.3 when checking the members. For example, the flexural buckling curves provided in 3-1-1/Figure 6.4 include all imperfections for a given member effective length of buckling. It should be noted that the equivalent geometric imperfections given in EN 1993 are not slenderness dependent, being a function of length only, whereas the imperfections in the resistance formulae are a function of the slenderness with a cut-off level such that below a certain slenderness, no imperfection is applied in order to replicate the results of tests for stocky elements. It should therefore be noted that if the compression resistance of a simple pin-ended member of a given slenderness is obtained using second order analysis with the imperfections given in Table 5.1 of EN 1993-1-1 for a particular buckling curve, the resulting resistance will usually be slightly lower than that obtained from the corresponding resistance curve in 3-1-1/Figure 6.4. For this reason, the UK NA to EN 1993-1-1 requires the following:

For elastic analysis of the cross-section, the initial imperfections for an individual section about a particular axis should be back-calculated from the formula for the buckling curves given in BS EN 1993-1-1:2005, ***6.3*** *using the elastic section modulus.*

It may not be immediately apparent to designers how to do this but in fact 3-1-1/5.3.1(11) itself provides guidance through an alternative method. To overcome this moderate conservatism caused by the difference between imperfections recommended for global analysis and those used in the resistance curves, EN 1993-1-1 provides an alternative method whereby the imperfection for the whole structure (global and local imperfections) or an element is based on the shape of the critical elastic buckling mode and with a magnitude directly relating to that used in the resistance curves for the particular slenderness. This unique imperfection is given by:

$$\eta_{\mathrm{init}} = \frac{\alpha\left(\bar{\lambda}-0.2\right)}{\bar{\lambda}^2} \frac{1-\dfrac{\chi\bar{\lambda}^2}{\gamma_{\mathrm{M1}}}}{1-\chi\bar{\lambda}^2} \frac{M_{\mathrm{Rk}}}{EI\eta''_{\mathrm{cr,max}}}\eta_{\mathrm{cr}} \qquad \text{(D5.3-1)}$$

η_{cr} represents the local ordinates of the mode shape and η'' is the curvature produced by the mode shape such that $EI\eta''_{cr,max}$ is the greatest bending moment due to η_{cr} at the critical cross section. Other terms are as follows:

α is the imperfection factor taken from 3-1-1/Tables 6.1 and 6.2 for the relevant mode of buckling. For varying cross section, the greatest value can conservatively be taken.

$\bar{\lambda} = \sqrt{\dfrac{\alpha_{ult,k}}{\alpha_{cr}}}$ where $\alpha_{ult,k}$ is the load amplifier to reach the characteristic squash load N_{Rk} of the most axially stressed section and α_{cr} is the load amplifier for elastic critical buckling.

χ is the reduction factor for the above slenderness determined using the relevant buckling curve appropriate to α.

The derivation of this equation is given in reference 3.

This method and the proposed modification in the UK NA have the disadvantage that the slenderness of the structure has to be determined first from an eigenvalue analysis which tends to reduce the appeal of second order analysis as a practical design method. Second order analysis of a pin-ended member with imperfections determined in this way will however produce the same resistance as obtained from the resistance curves.

The above discussions relate in the main to flexural buckling**.** If lateral torsional buckling is to be taken into account by second order analysis, the compression flange can be given a bow imperfection about the beam minor axis. A value of 0.5 e_0 is recommended in 3-1-1/5.3.4(3) where e_0 is again taken from 3-1-1/Table 5.1 (or back-calculated according to the UK NA, which will improve the resistance) but the UK NA modifies this to the full value of 1.0 e_0.

Example non-linear analysis for global buckling

The same FE model of paired girders discussed in section 2.1 above was analysed under the same loading considering non-linear material properties including strain hardening in accordance with 3-1-5/Annex C (and in this case including the partial material factor for steel) and non-linear geometry and including an initial deformation with shape corresponding to the first elastic global buckling mode. This was used to determine the collapse load. The magnitude of the largest bow deflection in this mode was taken as $L/150$ for curve d of Table 5.1 of EN 1993-1-1. The maximum moment reached and the moment at which first yield occurred were noted. Failure occurred by rotation of the braced pair over a span in the same shape as the elastic buckling mode of Figure 3; this equivalence in shape between eigenmode and ultimate collapse mode will not generally occur in all buckling problems. Where there is not equivalence, a refined (lower) prediction of the ultimate load will usually be obtained by using the collapse geometry as a revised imperfection geometry for a new analysis.

Figure 9 shows the load-deflection curve up to failure for the bridge. The ultimate resistance obtained by this method is given in Table 1 above. Non-linear analysis can be used to extract greater resistance from beams for a number of reasons which include benefit from:

- partial plastification of the tension zone in non-compact sections
- strain hardening
- moment redistribution in statically indeterminate structures (but not in the above example).

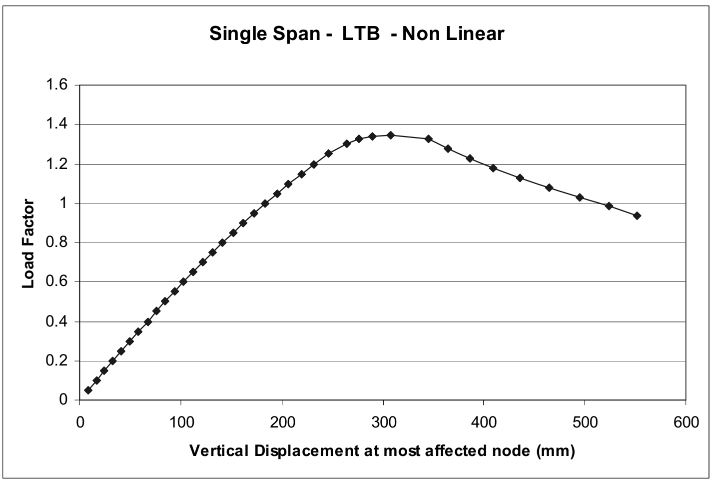

Figure 9. Load-deflection curve for non-linear analysis of single span model

Local buckling

Analysis of local buckling problems often requires a greater degree of experience and understanding, particularly in the application of imperfections.

3-1-5/C.5 gives guidance on imperfections for the local modelling of plate elements. In general, the distribution (or shape) of the imperfections to be used can be determined by one of four methods:

1) Using the same distribution as the mode shapes found from elastic critical buckling analysis

Elastic critical buckling analysis can be used to determine a unique imperfection distribution, with the same form as the buckling mode shape, in the same manner as discussed in section 3.1 above for frames. It is often assumed that this method of applying imperfections will maximise the reduction in resistance but this is not always true and there are difficulties in implementation. The imperfection distribution will vary with each load case and it is difficult to specify the imperfection magnitude for coupled modes involving both overall stiffened panel buckling and local sub-panel buckling. The elastic buckling mode with the lowest load factor may not also be the critical mode shape for reducing ultimate strength. Often, a slightly lower resistance is produced using method 4).

2) Using assumed imperfection shapes based on buckling under direct stress

The imperfection distribution can be based on the local and global plate buckling mode shapes for compression acting alone in the longitudinal direction. This method will not necessarily maximise the loss of resistance, but the resulting resistance will usually not be far from the true resistance.

3) Applying transverse loading

A variation on 2) above is to apply transverse loading so that the first order effects of such loading replicate the first order effects of imperfections.

4) Application of the deformed shape at failure

In this method, the deformed shape of the structure obtained at failure from a previous analysis is used as the initial imperfection shape. This frequently gives the lowest resistance (but rarely significantly lower than the other methods). It has the disadvantage that the method is iterative, as an initial analysis to failure is required to produce the imperfection shape.

A more detailed description of a typical application of non-linear analysis to a local buckling problem (buckling of transversely stiffened webs in bending and shear) is given in reference 6.

Conclusions

Eurocode 3 offers some real improvements in the codification for the design of steel plate structures in that it provides a framework in which advanced methods may be used. This paper has shown that there are some significant benefits that can be realised from this approach but that there are also warnings that the more advanced methods require a greater level of understanding of the fundamental physics behind the code. There are risks for the unwary in unlocking the benefits of the code. Like all codes and standards, there is a reasonable expectation that the user is familiar with the subject matter and competent in its use.

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Appendix A – Overview of Analysis Approaches

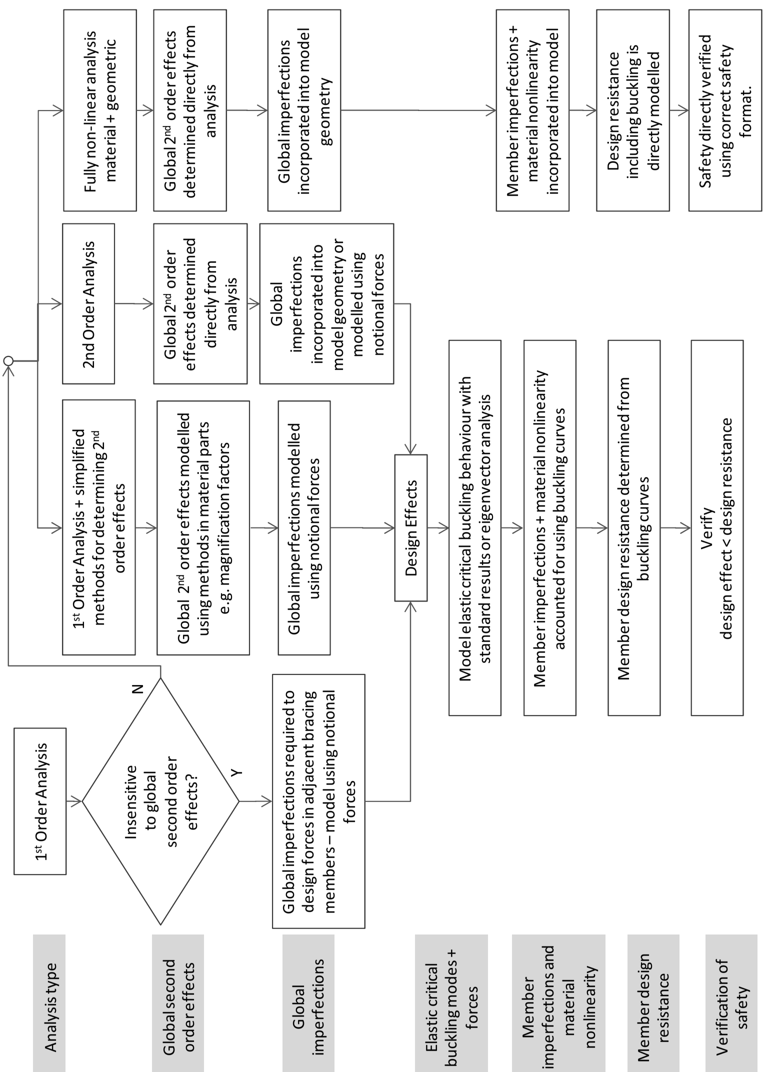

Figure A.1. Overview of analysis options to account for global and member buckling, second order effects and imperfections

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BS EN 1090-2:2008 AND PD 6705-2:2010

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Abstract

The paper gives a brief summary of the development of UK specifications for materials and workmanship for steel bridges including BS EN 1090-2 which represents a major change in UK practice. One of the main features of the new specification is the extensive list of choice items and additional information on which the specifier needs to make technical decisions. To assist in this process BSI is in the process of publishing PD 6705-2 which is described briefly in the paper.

Background to Development of UK Material and Workmanship Specifications for Bridges

British engineering has been at the forefront of the development of iron and steel bridge construction for nearly two centuries and there are many thousands of examples all over the world still doing good service after more than a century. This is a testament to both the design and the construction standards on which these bridges were based.

The first British Standard for steel girder bridges BS 153 was published in 1922 in two volumes containing five parts, namely: Part 1 – Materials and Part 2 – Workmanship, in the first volume[1] and Part 3 – Stresses, Part 4 – Details of construction and Part 5 – Erection in the second volume[2]. These standards covered riveted and bolted girders made with mild steel. Revisions of these two volumes were made in 1933 and 1937 respectively.

The most extensive amendments were made in the 1950s with the regrouping of the Parts and the inclusion of specifications for railway and highway loading. The Parts were re-grouped into three volumes; the first containing Part 1 – Materials and workmanship and Part 2 - weighing, shipping and erection (1958)[3], the second containing Part 3A loads (1954)[4] and the third containing Part 3B stresses and Part 4 design and construction (1958)[5]. Thus all specifications for requirements for fabrication and erection (Parts 1 and 2) were contained in one volume. Part 4 covered design of constructional details and not manufacturing aspects.

The extent of interest in this standard at that time can be judged by the fact that 45 Government departments and scientific and industrial organisations were involved in its supervision and/or preparation. The main new features of the 1958 version of Parts 1 and 2 and subsequent revisions in the 1960s were clauses dealing with high strength friction grip bolts and arc welding. These made extensive use of the many new British Standards which had been published for these methods of construction in the 1950s and 1960s.

In the early 1970s the buckling failures of various box girder bridges highlighted the limitations of the BS 153 design and specification Parts, where used for this new form of construction, as opposed to plate girders (for which BS 153 was written).

The resulting extensive research carried out in the early 1970s revealed the importance of welding residual stresses and geometrical imperfections in affecting buckling resistance of panels and stiffeners. The scope of BS 153 was not extended to cover box girders. Instead new execution requirements were published in the Interim Design and Workmanship requirements for the box girders which arose from the Merrison Committee inquiry and its subsequent recommendations.

At the end of the 1970s work on redrafting the new limit state bridge design code BS 5400 was completed. This took into account all the recent work on box girder design and manufacture and included that form of construction. In 1980, BS 153 Parts 1 and 2 were superseded by BS 5400-6[6] which included references to important new welding standards, BS 5135[7] (welding practice) and BS 4870-1[8] (weld procedure approval). BS 5135 contained extensive rules for the avoidance of hydrogen cracking, which was highlighted in the catastrophic failure of the Kings Bridge plate girders in 1962. BS 5400-6 also included detailed requirements for geometrical tolerances.

In the 1980s concerns were expressed by some fabricators over difficulties experienced in meeting the strict weld quality standards in BS 4870 consistently, which were also imposed for normal production welds. The BS bridge committee set up an investigation into the matter and after a lengthy examination of the fracture mechanics implications and the detection and measurement capability of commercial non-destructive testing (NDT) techniques, ultrasonic testing (UT) in particular, it was agreed that relaxations from the quality required for procedure approval could be justified for production using fitness-for-purpose (FFP) principles, particularly where fatigue stressing was not high. On the other hand it was concluded that there were difficulties of assuring in practice by non destructive means that the quality of workmanship in production, of welding in particular, was adequate to meet the stringent requirements needed when fatigue stressing was high. In 1999 BS 5400-6 and BS 5400-10[6] were revised to take account of these findings. BS 5400-6 included three levels of weld quality depending on the degree of fatigue stressing. BS 5400-10 provided designers with a method for assessing which quality was necessary, (in terms of the fatigue stressing), and also amended the detail classification tables to warn when the higher classifications might not be reliable unless special attention was paid to NDT procedures.

The results of this work were used to produce FFP based weld quality standards for other applications. This included BS 5950-2 for buildings[9] and ISO 10721 - the new International Standard for structural steelwork, Part 1 covering materials and design and Part 2 covering fabrication and erection[10].

BS 5400-6 was withdrawn by BSI in April 2010 in line with CEN requirements and is replaced by the new European based standard BS EN 1090-2. This ended nearly 90 years of development of standards reflecting best British practice for steel bridge manufacturing.

Background to Development of European Execution Requirements for Bridges

The CEN technical committee responsible for development of the Eurocodes for structures design CEN/TC250 was not responsible for the development of the associated specifications for materials and workmanship (now referred to as 'Technical requirements for execution' in Euro-English parlance). This separation of responsibilities represents a major difference from British practice.

When the draft Eurocode for steel structures DD ENV 1993-1-1[11] was published in 1992 reference was made to such European product standards as had already been published by CEN. The standard for 'Execution of steel structures' was at that time without an EN number. A different CEN Committee, CEN/TC135 was set up in the late 1980s to draft the execution standard for structural steelwork, which was eventually published by CEN as ENV 1090[12] in six parts, starting with Part 1 in 1996 which gave 'general rules and rules for buildings'. The five other parts were published over the next four years including Part 5 which gave supplementary rules for the execution of bridges. The UK was well represented on the CEN working groups preparing these different parts. The ENVs were subject to two year Enquiry Periods after which National Standards Bodies submitted comments.

On receipt of the comments CEN/TC135 decided to merge all six parts into one common part, which became EN 1090-2[13] under one working group. Two other working groups were set up to produce a new Part 1[14] on certification of conformity and a new Part 3[15] covering the execution of aluminium structures. The UK was active in all three working groups. Thus the requirements for the execution of steel bridges were no longer separated from other structural steel applications as they were in the British Standards.

Throughout the whole process of developing EN 1090-2 it was clear that there were fundamental differences in practice across Europe and that these could only be resolved by introducing a wide selection of choice items and leaving many gaps to be filled in by the user. A further factor affecting the development of the standard was that there was no funded Project Team for the drafting work, as there had been with all the Eurocode parts. These factors inevitably had a delaying effect on the programme, with the result that the final Enquiry draft pr EN 1090-2[16] was only put out for comment in 2005 by which time many of the key parts of Eurocode 3 had been finalised. This was in spite of the fact that EN 1993-1-1[17] made it clear that the validity of the design rules were dependent on execution of the work being in accordance with EN 1090. EN 1090-2 was essential for defining the necessary quality and testing standards which would have a significant effect on the reliability of the design resistance values (as determined by the various γ_M values which had already been prescribed by each member state in their National Annexes to EN 1993).

A further novel feature which was introduced into pr EN 1090-2 and which did not appear in ENV 1090, was that of 'Execution Class'. This was a device to enable flexibility to be introduced, due to the wide range of potential applications. This effectively created a set of widely differing standards within the document which could be selected by the user either for the whole structure or different parts of it. Informative advice on the selection of execution class was included, involving a matrix of different parameters which included three levels of 'consequence class' (CC), which in BS EN 1990[18] are linked to three classes of reliability (RC), each with its own target reliability index. A further parameter was the new 'Service Category' (SC) which was only defined in qualitative terms, (eg whether or not the structure or components were deemed to be 'susceptible' to fatigue).

The BS Bridge Committee was very active in preparing comments on pr EN 1090-2, which included fundamental concerns about the new Execution Class matrix and its consistency with the reliability principles in BS EN 1990. They also prepared proposals for clauses to address their concerns and to improve the compatibility with BS EN 1990. The comments included proposals for specific testing and fitness-for-purpose assessment of welds, based on principles used BS 5400-6 and BS 5950-2. These comments were considered by the BS Committee responsible for BS 5950-2 to be relevant to steel structures in general and were included as such with all the UK Enquiry comments.

Whilst many of the UK comments were accepted by CEN/TC135, the UK's fundamental concerns about the use of Execution Class and the need to have specific fitness-for-purpose testing requirements were not, and no reasons for their rejection were provided. This was a major contributory factor leading to the UK casting a negative vote to the final version of pr EN 1090-2. Sweden and Finland, who had also been very active in the CEN working group also voted negatively, but the final weighted vote was still sufficient to approve it for publication.

The resulting BS EN 1090-2:2008[19] now supersedes BS 5400-6 and BS 5950-2. It represents a substantial change in specification of materials and workmanship for steel structures in general not only bridges. The new standard is considerably longer than its predecessors and includes many more clauses requiring user decision as can be seen from the table below.

Specification	Reference no.	BS 153: Parts 1 and 2	BS 5400-6	BS EN 1090-2
	Version, year	1958	1999	2008
Number of pages		25	38	203
Size – paper version		A5	A4	A4
Clauses requiring User decision		20	39	200
Normative references		25	65	177
Normative content		100%	90%	95%

Table 1. Developments in scale of British and European based materials and workmanship specifications for steel bridges over the last half century

The main benefits of BS EN 1090-2 over its predecessors are as follows:

- It is up to date with all the new BS EN reference standards which have emerged over the last 20 years.
- It addresses higher strength steels, cold rolled hollow sections and cables.
- It addresses laser and plasma cutting.

- It includes qualification requirements for welding co-ordinators, welders and weld inspectors.
- It addresses quality management systems, including documentation, plans etc which lays the basis for approval of factory production control (FPC) systems of fabricators.
- It includes a wider range of fastener types and methods of tightening of preloaded fasteners.
- It covers a wider range of geometrical tolerances.
- It covers surface treatment for corrosion protection.

The main disadvantages of BS EN 1090-2 as far as the specifier and/or purchaser are concerned are as follows:

- It does not set a clear cut standard, as its predecessors did. It contains a range of standards.
- Selection of an inappropriate standard by a user can result in a bridge whose structural integrity could either be seriously at risk or could be unnecessarily expensive to produce.
- The informative guidance on selection of Execution Class is based on a premise that the UK objected to on grounds that it was technically unsound (as discussed above).
- The user frequently has to supply additional information for which he/she may not have sufficient experience to undertake with confidence.
- Control of weld quality relies entirely on one of three choices of FPC systems. The extent and methods of weld testing are very loosely defined and not specific to each bridge. The acceptance levels are arbitrary and are not based on FFP criteria. These are major differences from BS 5400-6:1999.

As a consequence the BS Bridge Committee agreed to develop a published document (PD) to assist specifiers, employing BS EN 1090-2 for steel bridge work, in making sound choices when selecting their technical requirements. This document has been approved for publication by BSI and is described below.

PD 6705-2 Recommendations for the Execution of Steel Bridges to BS EN 1090-2

PD 6705-2[20] was prepared by the working group of the BS Bridge Committee responsible for maintaining BS 5400-6 and its successors. The working group has been closely involved with the developments of EN 1090 over a period of 20 years and has included design experts closely involved with EN 1993, bridge procurement agencies, and steel fabricators.
PD 6705-2 has therefore benefitted from an intimate knowledge of the main technical aspects in both BS 5400-6 and BS EN 1090-2, including the fundamental issues raised by the UK at the 2005 Enquiry stage.

The prime objective of PD 6705-2 is to provide users with sufficient technical advice that they can make appropriate choices and supply essential additional information. It is not intended to supply ready-made specification clauses for immediate verbatim addition to a contract specification, although some of the material may be transferred across with appropriate editorial adjustment.

PD 6705-2 only addresses key technical user decision issues which have a significant effect on the structural integrity of steel bridges. This applies primarily to the resistance of members and joints to buckling, fracture and fatigue failure, and is intended to ensure that the controls on quality are consistent with the γ_M values selected by the UK in BS EN 1993 (including the various PDs referenced in the National Annexes). PD 6705-2, as a consequence, addresses approximately a third of the total 200 user decision items in BS EN 1090-2, (some of which do not relate to bridges, others do not have a significant effect on structural integrity and others do not need additional information).

The main features of PD 6705-2 are:

- It is intended to cover all types of bridge from the smallest and simplest footbridge to the most sophisticated and complex structure in high tensile steel with highly dynamic loading.
- It is not intended to cover contractual matters, only technical ones.
- The user need only address those recommendations which are relevant to the application(s) in hand.
- It provides information on the background to the development of EN 1090-2 and its main differences from BS 5400-6.
- It provides explanatory notes on the background to many of the recommended choices and additional information.
- It extends the concept of a service category from a qualitative one to a quantified one, whereby the performance requirements are clearly defined in terms of specific levels of static and cyclic stressing, using principles similar to those use in BS 5400-6 and BS 5400-10. The term ‘quantified service category’ has also been used in PD 6705-3:2009[21] which provides guidance on the use of BS EN 1090-3:2009 [22] (execution of aluminium structures), and which uses similar principles to those in BS 8118-1 and -2[23] (now superseded by EN 1999[24] and BS EN 1090-3 respectively).
- Tables giving specific methods and scope of testing of welds and simplified FFP based acceptance criteria for each method are given for the various quantified service categories. These are similar in layout to those in PD 6705-3. The FFP based acceptance criteria provide definite rejection limits, which are not defined in BS EN 1090-2.

It is expected that there will be many specifications written in future for a wide variety of bridge applications, all based on BS EN 1090-2. These are likely to vary from the very simple to the very detailed. Some may be project specific, others may be for generic types of bridge or be intended to embrace the full range of bridge applications. The degree of refinement of the resulting specification based on BS EN 1090-2 and PD 6705-2 can be decided by the user. The way in which the guidance in PD 6705-2 is used to construct these specifications will therefore vary according to the appropriate balance needed between simplicity of application and avoidance of unnecessary overspecification.

Conclusion

BS EN 1090-2 represents a very substantial change in UK practice in terms of specification of materials and workmanship for steel bridges. PD 6705-2 is considered to be an essential aid to those using the new European Standard for preparing specifications.

Acknowledgements

The authors would like to acknowledge the extensive contributions of the members of Working Group 6 of the BS Bridge Committee B/525/10 over many years during the development periods of EN 1090 and PD 6705-2.

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EN 1994 – COMPOSITE AND EN 1995 – TIMBER

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EN 1994-2: OVERVIEW OF COMPOSITE BRIDGE DESIGN, THE UK NA FOR EN 1994-2 AND PD 6696-2

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Abstract

The objective of this paper is to give the background to the development of the National Annex to BS EN 1994-2 and the development of the provisions of *PD 6696-2: Background paper to BS EN 1994-2 and the UK National Annex to BS EN 1994-2*. In producing the National Annex, the relevant clauses in the Eurocodes were reviewed, comparing them against clauses in the UK bridges standard BS 5400 and other published national and international standards. Where significant discrepancy with existing practice was revealed, further justification was sought from other research material and the Eurocode clauses were then accepted or amended to suit. The PD document was prepared with two primary objectives in mind:

(i) Provision of information on topics not covered by EN 1994.
(ii) Provision of guidance where it was considered further explanation of the Eurocode provisions was desirable for their correct and consistent application.

The source of much of this material was the *Designers' Guide to EN1994-2, Eurocode 4: Design of composite steel and concrete composite structures. Part 2: General rules and rules for bridges*, but other sources were also used.

Introduction

The objective of this paper is to give the background to the development of the National Annex to BS EN 1994-2 and the development of the provisions of *PD 6696-2: Background paper to BS EN 1994-2 and the UK National Annex to BS EN 1994-2*. In producing the National Annex, the relevant clauses in the Eurocodes were reviewed, comparing them against clauses in the UK bridges standard BS 5400 and other published national and international standards. Where significant discrepancy with existing practice was revealed, further justification was sought from other research material and the Eurocode clauses were then accepted or amended to suit. The PD document was prepared with two primary objectives in mind:

(i) Provision of information on topics not covered by EN 1994.
(ii) Provision of guidance where it was considered further explanation of the Eurocode provisions was desirable for their correct and consistent application.

The first item was the subject of debate because the principle-based approach used in the Eurocodes together with the wide range of analytical techniques permitted combine to ensure that it is usually possible to design all elements of a bridge utilising Eurocode methodology. It was considered to be undesirable to *require* an increase in the level of complexity of analysis over and above that used in previous practice, although the flexibility to *permit* such analysis was considered to be beneficial. The material included under (i) is therefore usually in the form of design rules that can be applied by hand methods of calculation with a similar level of complexity as required by previous practice to BS5400.

The remainder of this paper looks at each NA and PD entry in turn and provides explanation for the particular requirements given for the clauses which are not self-explanatory. References to clauses in EN 1994-2 have been abbreviated below. For example, 4-2/5.5.2.4(2) is a reference to clause 5.5.2.4(2) of EN 1994-2. It should be noted that the clause numbering in the NA and PD do not follow that in EN 1994-2.

NA to EN 1994-2

Shear Connection (NA cl. 2.1)

EN 1994-2 provides resistances and detailing rules for headed stud connectors only. It is sometimes however convenient to provide other types of connectors where a high local longitudinal shear resistance is required, such as at the end of beams in integral bridges. The commonest connector in use in this situation is the block and hoop connector. Design rules are given in BS 5400-5[1] and it was therefore considered that it would be useful to provide design guidance on such connectors within the UK National Annex to EN 1994-2.

4-2/6.6.1.1(12) permits the use of other types of connector provided that the behaviour assumed in design is “based on tests and supported by a conceptual model”. It is therefore not possible to simply lift the design rules from BS 5400-5 because of the lack of conceptual model. The method provided in PD6696-2 (discussed below) satisfies clause 6.6.1.1(12) because it is derived from a conceptual model and is supported by tests. The BS5400-5 requirements themselves are test-based and the formula derived for the National Annex leads to more conservative values of resistance.

Combination Factor for Global and Local Action Effects (NA cl. 2.5)

The NA sets the combination factor for global and local action effects for bridges other than road bridges as 1.0 and refers to the National Annex to BS EN 1993-2:2006 for the equivalent factor for road bridges. The latter also provides a factor of 1.0 and hence there is consistency across both rail and road bridges.

Choice of the Methods for Calculating Elastic Resistance to Bending (NA cl. 2.6)

Class 4 cross sections can be treated either in accordance with the effective section method of 3-1-5/4 or the reduced stress method in 3-1-5/10. EN 1993-2 allows a nation to choose either one or both methods for use in steel design and EN 1994-2 allows the same choice for steel-concrete composite bridges. The UK NA to EN 1993-2 permits both methods to be used and therefore the NA to EN 1994-2 also permits both methods for compatibility. The papers on the background to the NAs for EN 1993-2 and EN 1993-1-5 provide more explanation of the methods.

Values of $C_{Rd,c}$ and k_1 for Concrete Flanges in Tension (NA cl. 2.7)

In a continuous composite beam-and-slab bridge, the resistance of the deck slab to vertical shear (caused, for example by a wheel load) is influenced by the longitudinal force in the concrete deck. For this subject, clause 6.2.2.5(3) of EN 1994-2 refers to clause 6.2.2 of EN 1992-2. That clause gives an expression for design ultimate shear stress that is, essentially,

$$v_{Rd,c} = C_{Rd,c}\,f(d, \rho_1, f_{ck}) + k_1\sigma_{cp}, \qquad \text{from (6.2a) of EN 1992-2}$$

where σ_{cp} is the axial force per unit area of concrete, with compression positive, and the first term is the usual expression for shear resistance in a reinforced concrete member not requiring design shear reinforcement. It is a function of effective depth d, tensile reinforcement ratio ρ_1 and concrete strength f_{ck}. It should be noted that the reinforcement ratio, ρ_1, relates only to the reinforcement in tension under the loading which produces the shear force. Thus, for shear from a wheel load, only one layer of reinforcement (top or bottom as appropriate) should be considered and the effective depth, d, will relate to this reinforcement.

The values for the constants recommended in EN 1992-2 are:

$C_{Rd,c} = 0.18/\gamma_C$ and $k_1 = 0.15$.

These recommended values are adopted by the NA to EN 1992-2. Equation (6.2a) is principally intended to allow for the *favourable* influence of prestress or other compression force on shear resistance, but also reduces shear strength in the presence of tensile force in the slab as occurs in main beam hogging regions. The example in Tables 1 to 3 below shows that when applied to the concrete deck slab of a hogging moment region of a composite bridge, σ_{cp} could be so large (and negative) that the shear resistance $v_{Rd,c}$ was below zero. This is clearly incorrect; tensile forces can reduce shear strength but not to negative values. Also BS 5400 Part 4, for example, does not reduce the resistance to vertical shear when in-plane tension is present and bridges have been satisfactorily designed this way for many years.

The two constants in the equation above are again nationally determined parameters in EN 1994-2, so it was possible for EN 1994-2 to recommend different values to EN 1992 for use where a concrete flange is in tension.

The recommended values in EN 1994-2 clause 6.2.2.5(3) are:

$C_{Rd,c} = 0.15/\gamma_C$ and $k_1 = 0.12$.

With the slightly reduced values in EN 1994-2, resistance to vertical shear would still depend on the co-existing hogging bending moment in the composite beam, if it were not for the important further recommendation in the Note to clause 6.2.2.5(3). When first published in 2005, the note stated:

'Also where the stress σ_{cp} is tensile and exceeds $\sigma_{cp,0}$, it should be replaced by $\sigma_{cp,0}$ ……with the recommended value $\sigma_{cp,0}$ = 1.85 N/mm^2.'

Care is however needed with sign as σ_{cp} has the sign convention of compression positive so the wording in EN 1994-2 was clarified in PD 6696-2 to say:

'Also where the stress σ_{cp} is tensile and is less than $\sigma_{cp,0}$, it should be replaced by $\sigma_{cp,0}$ ……with the recommended value $\sigma_{cp,0}$ = - 1.85 N/mm^2.'

This correction is no longer necessary in PD 6696-2 as it has been made in the Eurocode itself via the July 2008 corrigendum.

For a concrete flange with a total of 1% of longitudinal reinforcement (corresponding approximately to two layers of 16 mm diameter bars at 150 mm centres in a 250 mm deep deck slab), this value is reached when the mean stress at ULS in the reinforcement (both layers) is only 185 N/mm^2. For 2% of reinforcement, this reduces to 93 N/mm^2. Stresses in practice usually exceed these limits at the ultimate limit state.

It follows that normally, σ_{cp} can be taken as -1.85 N/mm^2, which generally removes the interaction noted above. The design shear strength of the concrete in the slab is then reduced by 0.12 × 1.85 = 0.22 N/mm^2 on planes normal to the direction of longitudinal (and/or transverse) tension. This also affects verifications for punching shear.

The method of EN 1994-2 and the recommended constants are identical to those of equation (4) of reference 2. Although prestress makes this less of an issue in many concrete bridges, it is apparent that the behaviour of deck slabs in concrete and composite bridges behave the same. It is to be hoped that EN 1992-2 and EN 1994-2 are brought into line in later amendments.

There are several differences between the rules of clause 5.4.4.1 of BS 5400 Part 4 : 1990 and those of ENs 1992-2 and 1994-2. For typical bridge-deck slabs in tension, the BS gives the higher resistance to vertical shear, but there is a cut-off at f_{cu}= 40 MPa, (f_{ck} = 32 MPa), whereas the resistance to the Eurocodes is proportional to $f_{ck}^{1/3}$ for concretes up to grade C50/60 (dictated by the National Annex to EN 1992-2). The examples in Tables 1, 2 and 3 illustrate that the recommendations in EN 1994-2 are conservative and that those in EN 1992-2 lead to unreasonably low values of shear resistance other than for very small global reinforcement tensile stresses. In all Tables, the deck slab is 300 mm thick, with effective depth 250 mm. In Tables 1, 2, and 3, σ_{cp} = -1.85 N/mm^2, -3.70 N/mm^2 and -7.40 N/mm^2 respectively. The reinforcement ratio, ρ_1, relates to the reinforcement in one face of the slab only. The reinforcement tensile stress corresponding to these values of σ_{cp} is given in the tables. Table 1 does not represent very realistic reinforcement tensile stresses, but corresponds to the stress for which further increases in tension cause no further reduction of shear resistance. EN 1992-2 is most conservative in Tables 2 and 3 and gives negative resistances in Table 3.

	Reinforcement stress (MPa)	BS 5400-4	EN 1994-2	EN 1992-2
$\rho_1 = 0.01, f_{ck} = 32$ MPa	111	0.88	0.60 - 0.22 = 0.38	0.72 – 0.28 = 0.44
$\rho_1 = 0.02, f_{ck} = 50$ MPa	56	1.11	0.88 - 0.22 = 0.66	1.06 – 0.28 = 0.78

Table 1. Design shear strengths of concrete slab in vertical shear, MPa with σ_{cp} = -1.85 MPa

	Reinforcement stress (MPa)	BS 5400-4	EN 1994-2	EN 1992-2
$\rho_1 = 0.01, f_{ck} = 32$ MPa	222	0.88	0.60 - 0.22 = 0.38	0.72 – 0.56 = 0.16
$\rho_1 = 0.02, f_{ck} = 50$ MPa	111	1.11	0.88 - 0.22 = 0.66	1.06 – 0.56 = 0.50

Table 2. Design shear strengths of concrete slab in vertical shear, MPa with σ_{cp} = -3.7 MPa

	Reinforcement stress (MPa)	BS 5400-4	EN 1994-2	EN 1992-2
$\rho_1 = 0.01, f_{ck} = 32$ MPa	444	0.88	0.60 - 0.22 = 0.38	0.72 – 1.12 = -0.4
$\rho_1 = 0.02, f_{ck} = 50$ MPa	222	1.11	0.88 - 0.22 = 0.66	1.06 – 1.12 = -0.06

Table 3. Design shear strengths of concrete slab in vertical shear, MPa with σ_{cp} = -7.4 MPa

As a result of the above analysis and comparisons it was proposed that the National Annex to EN 1994-2 should follow the recommendations of the Note to clause 6.2.2.5(3).

Shear Connection (NA cl. 2.9)

4-2/6.6.1.1(13) identifies a problem that occurs adjacent to cross frames or diaphragms between beams. For multi-beam decks, beams are often braced in pairs such that the bracing is not continuous transversely across the deck. The presence of bracing locally significantly stiffens the bridge transversely. Moments and shears in the deck slab are attracted out of the concrete slab and into the bracing as shown in Figure 1 via the transverse stiffeners. This effect is not modelled in a conventional grillage analysis unless the increased stiffness in the location of bracings is included using a shear flexible member with inertia and shear area chosen to match the deflections obtained from a plane frame analysis of the bracing system. Three-dimensional space-frame or finite-element representations of the bridge can be used to model these local effects more directly.

The transfer of moment causes tension in the shear connectors on one side of the flange and induces compression between concrete and flange on the other. Welds at tops of stiffeners must also be designed for this moment, which often leads to throat sizes greater than a ‘nominal’ 6 mm. In composite box girders, similar effects arise over the tops of the boxes, particularly at the locations of ring frames, bracings or diaphragms.

Although invited to provide additional guidance, the UK NA does not do so as the problem and guidance necessary is very bridge type-specific.

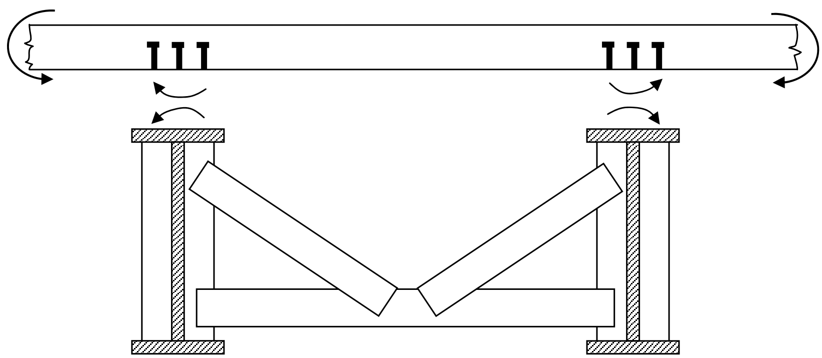

Figure 1. Example of bending moments from a deck slab attracted into bracings

Specific Measures to Limit the Heat of Hydration of Cement and the Temperature Difference to Be Considered (NA Cl. 2.14)

4-2/7.4.1(6) draws attention to the need to control cracking caused by early thermal shrinkage. The heat of hydration causes expansion of the concrete before it is stiff enough for restraint from steel to cause much compressive stress in it. When it cools, it is stiffer, so tension develops. This can occur in regions that are in permanent compression in the finished bridge and may require crack-control reinforcement for this phase only. The check is made assuming that the temperatures of the steel and the concrete are both uniform. The concrete is colder, to an extent that may be given in the National Annex.

The recommended temperature difference in EN 1994-2 is 20 K but the NA increases this to 25 K based on the temperature rise data in BD 28/87[5].

Shear Connection and Transverse Reinforcement NA.2.15

4-2/8.4.3(3) allows the spacing rules for individual connectors to be relaxed if connectors are placed in groups but many of the deemed-to-satisfy rules elsewhere in EN 1994-2 then no longer apply. The designer then has to explicitly consider the relevant effects from first principles, utilising other application rules in the Eurocodes where possible. There are many effects to be considered but the one referred to in clause 8.4.3(3) is the need to check the local resistance of the slab to the concentrated force from the connectors and provide local transverse reinforcement as required. Groups of studs apply a force analogous to that from an anchorage of a prestressing cable. The region of transverse tension does not coincide with the location of the group and both the quantity and the location of the transverse reinforcement required may differ from that given by 4-2/6.6.6. The strut and tie and partially loaded area rules of EN 1992-1-1 are relevant for the design of this reinforcement.

If connectors are arranged in groups there will be other design considerations in addition to that of transverse reinforcement including the effects of non-uniform shear flow, separation of slab and steel (potentially leading to corrosion) and buckling of the steel flange. Many of these will be very specific to the geometry being used and, as such, the NA does not attempt to provide further guidance. Reference 3 provides more advice and background.

PD 6696-2 Background to EN 1994 and UK National Annex Recommendations

Combinations of Local and Global Action Effects in Deck Slabs (PD cl. 3.4)

The Note to 4-2/5.4.4(1) refers to Normative Annex E of EN 1993-2. This annex was written for all-steel decks, where local stresses in welds can be significant and where local and global stresses always combine unfavourably. It recommends a combination factor ψ for local and global effects that depends on the span and ranges from 0.7 to 1.0. The application of this rule to reinforced concrete decks that satisfy the serviceability requirements for the combined effects is believed to be over-conservative, because of the beneficial local effects of membrane and arching action. By contrast, if the EN 1993 rules are adopted, global compression in the slab is usually favourable so consideration of the maximum compressive global stress when checking local effects may actually be un-conservative.

As a result, the PD recommends that the flexural effects of local and global loads in deck slabs in the longitudinal direction should be combined for serviceability verifications, but, where elastic global analysis is used, need not be combined for checks for ultimate limit states other than fatigue. This has proved to be an acceptable approach over the years when using BS 5400 Part 4. The PD also however requires a check on compressive stress at SLS in this instance, regardless of exposure class, because otherwise there is the potential that the effects of combined local and global compression are not considered at either limit state.

Classification of Cross-sections (PD cl. 3.5)

The guidance in the PD, requiring the section classification to be determined considering the actual combination of bending and axial compression, was included because simple linear interaction equations could otherwise allow the plastic bending resistance to be used when considering the bending moment contribution, when the presence of axial force might make the cross section non-compact, thus making the assumption of plastic bending behaviour invalid.

Elastic Resistance to Bending (PD cl. 4.1)

The PD offers a clarification that in calculating $M_{el,Rd}$, the stresses from the primary effects of shrinkage should be included, unless they may be neglected in accordance with BS EN 1994-2:2005, 6.2.1.5 (5). This is simply because, whilst this was intended in EN 1994-2, there is no explicit statement to that effect.

Resistance to Vertical Shear – Additional Rules for Beams in Bridges (PD cl. 4.3)

This additional requirement is no longer needed as it has been incorporated in EN 1994-2 itself. See the comments made under NA clause 2.7.

Bending and Vertical Shear (PD cl. 4.4)

The value of M_{Ed} for use in the interaction with Class 3 and 4 cross-sections was not clearly defined in the first published version of BS EN 1994-2. Clause 6.2.2.4(3) stated only that EN 1993-1-5 clause 7.1 was applicable '*using the calculated stresses of the composite section'*. There was no problem with interpretation in earlier drafts as η_1, the accumulated stress divided by the appropriate stress limit, was used in the interaction rather than $\bar{\eta}_1$. The PD therefore recommended that M_{Ed} be taken as the greatest value of $(\Sigma\sigma_i)W$, where $\Sigma\sigma_i$ is the total accumulated stress at an extreme fibre and W is the elastic modulus of the effective section at the same fibre at the time considered. This bending moment, when applied to the cross-section at the time considered, produces stresses at the extreme fibres which are at least as great as those accumulated. A further reason for this definition of M_{Ed} was that there is limited test justification for the approach of using the plastic bending resistance in the interaction with shear in unpropped composite construction.

After completion of the PD, EN 1994-2 was amended to define M_{Ed} as the total moment on the cross section. This means the existing PD clause is redundant.

Bending, Shear and Axial Force (PD cl. 4.5)

When axial force is present, some further interpretation of 3-1-5/7.1 is required for composite sections and this is given in the PD. It should be noted that the definition of M_{Ed} in the PD clause should be deleted for the reasons mentioned above under clause 4.4.

General Methods for Buckling of Members and Frames (PD cl. 4.6)

There are numerous methods presented in the Eurocodes for considering lateral and lateral torsional buckling of members subject to bending and axial load and the relevant clauses are distributed somewhat around the various Eurocode bridge parts. The PD has identified what the main approaches are and offered guidance on their application. The subject is too extensive to be covered in this paper and reference can be made to reference 3 for further guidance.

Influence of Tension on Shear Resistance (PD cl. 5.1)

Resistance of studs to higher tensile forces has been found to depend on so many variables, especially the layout of local reinforcement, that no simple design rules could be given in EN

1994-2. Relevant evidence from about 60 tests on 19-mm and 22-mm studs is presented in reference 4, which gives a best-fit interaction curve. In design terms, this becomes

$$(F_{ten} / 0.85\, P_{Rd})^{5/3} + (P_{Ed}/P_{Rd})^{5/3} \leq 1$$

Where the vertical tensile design force $F_{ten} = 0.1\, P_{Rd}$, this gives $P_{Ed} \leq 0.93\, P_{Rd}$, which is plausible. This expression was therefore provided in the PD. It should be used with caution, because some studs in these tests had ratios h/d as high as 9; but on the conservative side, the concrete blocks were unreinforced.

Design Resistance to Longitudinal Shear (PD cl. 5.3)

Neither EN 1992 nor EN 1994 deals with the case of longitudinal shear and coexistent transverse tension in the slab. This can occur in the transverse beams of ladder decks near the intersection with the main beams in hogging zones where the main beam reinforcement is in global tension. In such cases, there is clearly a net tension in the slab and the PD recommends that the reinforcement requirements for this tension should be fully combined with that for longitudinal shear.

Design Resistance of Block Connectors With Hoops (PD cl. 5.4)

The resistance to longitudinal shear, P_{Rd}, of steel blocks welded to a steel flange can be determined using EN 1992-1-1 clause 6.7. From EN 1994-2 clause 6.6.1.1(8), the resistance to uplift should be at least 0.1 P_{Rd} and this can be provided by the associated hooped reinforcing bars. None of the modes of failure involve interaction between concrete and steel, so their own γ_M factors can be used, rather than γ_V.

The connector block is assumed to be rigid and in order to realise this assumption, the height of the block should not exceed four times its thickness. An identical requirement was given in ENV 1994-1-1 clause 6.4.4(1). The longitudinal shear force may then be assumed to be resisted by a uniform stress, σ_{block}, at the face of each block. Lateral restraint enables this stress to exceed f_{cd} to an extent given in clause 6.7 of EN 1992-1-1, 'Partially loaded areas':

$$\sigma_{block} = F_{Rdu}/ A_{c0} = f_{cd} \sqrt{A_{c1}/A_{c0}} \leq 3.0\, f_{cd} \quad \text{(6.63) of EN 1992-1-1}$$

where A_{c0} is the loaded area and A_{c1} is the 'design distribution area' of similar shape to A_{c0}, shown in Fig. 6.29 of EN 1992-1-1.

Clause 6.7 requires the line of action of the force to pass through the centres of both areas, but in this application it can be assumed that the force from area $b_2d_2/2$ is resisted by the face of the block, of area $b_1d_1/2$, because the blocks are designed also to resist uplift. In this case therefore, A_{c0} is the area of the front surface of the block connector, equal to 0.5 b_1 d_1 and A_{c1} is the design distribution area at the rear surface of the adjacent connector, equal to 0.5 b_2 d_2.

The dimensions of area A_{c1} are fixed by clause 6.7 of EN 1992-1-1 as follows:

$$b_2 \leq b_1 + h \quad \text{and} \quad b_2 \leq 3b_1; \qquad d_2 \leq d_1 + h \quad \text{and} \quad d_2 \leq 3d_1$$

Additionally, no part of A_{c1} above the steel flange must lie outside the concrete slab.
The resistance of the block may be determined from:

$F_{Rdu} = P_{Rd} = A_{c0}\ \sigma_{block}$
Therefore:
$P_{Rd} = A_{c0}\sqrt{A_{c1}/A_{c0}}\ f_{cd} \leq 3.0\ A_{c0}\ f_{cd}$

The resistances produced from the above are lower than the equivalents from BS5400 Part 5.

If lightweight aggregate is used, clause 11.6.5 of EN 1992-1-1 should be used in the above derivation instead of clause 6.7. This leads to:

$P_{Rd} = A_{c0}\left[A_{c1}/A_{c0}\right]^{\rho/4400} f_{lcd} \leq 3.0\ (\rho/2200)\ A_{c0}\ f_{cd}$

where ρ is obtained from Table 11.1 of EN 1992-1-1.

EN 1992-1-1 clause 6.7 would strictly require vertical reinforcement to be provided to control splitting from the vertical load dispersal. However, as both push tests and practice have shown this reinforcement to be unnecessary even when using the higher resistances to BS 5400-5, vertical reinforcement need not be provided here.

The hoops and their connections to the block should be designed for an uplift force of 0.1 P_{Rd} in accordance with EN 1994-2 clause 6.6.1.1(8). The anchorage of the hoops into the concrete over their height h_r should comply with the requirements of clause 8.4.4(2) of EN 1992-1-1.

The welds between the block and the steel flange should be designed for the shear, tension, and bending moment arising from resisting a coexistent shear of P_{Rd}, applied at mid-height of the block, and an uplift of 0.1 P_{Rd} in accordance with EN 1993-1-8.

Fatigue of the welds should be checked in accordance with EN 1994-2 clause 6.8.6.2(2). Fatigue of the reinforcement, structural steel and concrete should be verified in accordance with clause 6.8.7.1 of EN 1994-2. The method of Annex A of EN 1993-1-9 should be used only where:

- the connectors are attached to the steel flange by welds that are within the scope of EN 1993-1-9;
- the fatigue stress ranges in the welds can be determined realistically;

The exponent m should then have the value given in EN 1993-1-9; m = 8 should not be used.

Acknowledgements
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DESIGN ILLUSTRATION – COMPOSITE HIGHWAY BRIDGES

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Abstract

Design of a composite highway bridge to the Eurocodes will require reference to at least 14 separate Parts of the Eurocodes, each with its appropriate National Annex. To illustrate many of the aspects of applying the necessary documents to the design of typical multi-girder and ladder deck bridge configurations, SCI has published a book with two worked examples. This paper presents an overview of some of the design aspects revealed in preparation of those examples.

Introduction

For the last 20 years, SCI has provided guidance to the designers of composite highway bridges. General guidance on best practice, based on the views of experienced senior designers, has been accompanied by guidance on the use of design standards, notably, in the past, on the use of BS 5400. The guidance has been illustrated by worked examples, presenting detailed calculations, with references to the relevant clauses of the standards. That guidance has now been updated with two new publications for design in accordance with the Eurocodes, one that offers general guidance (SCI publication P356)[1] and one that presents two worked examples (SCI publication P357)[2]. In P357, one example is a multi-girder bridge and the other is a ladder deck bridge. The preparation of the examples has revealed many of the aspects where design practice differs, to a greater or lesser degree, from that in accordance with BS 5400. This paper presents an overview of those aspects.

The Examples

In SCI publication P357, Example 1 is a two-span multi-girder deck bridge with integral abutments. The arrangement of the bridge is shown in Figure 1. Example 2 is a three-span ladder deck bridge, with a curved soffit to the main girders in all three spans. The arrangement of the bridge is shown in Figure 2.

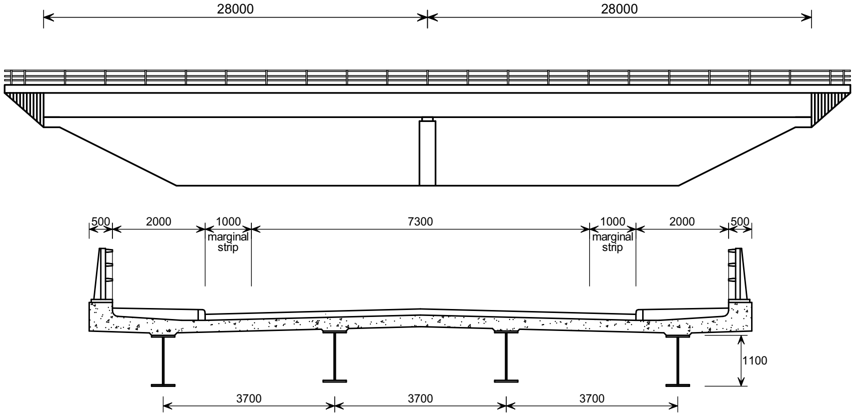

Figure 1. General arrangement of Example 1

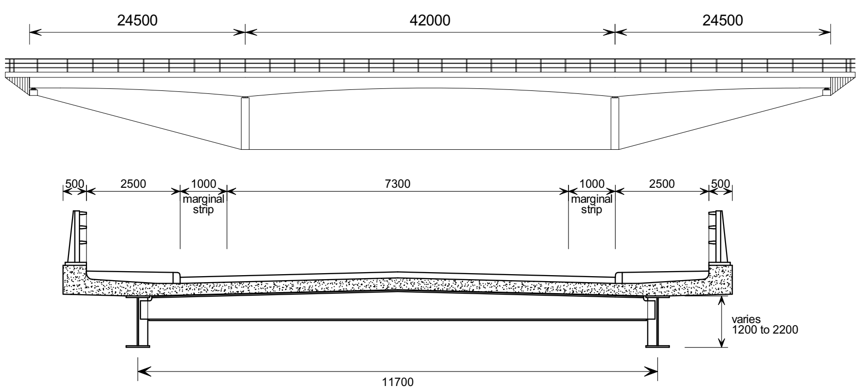

Figure 2. General arrangement of Example 2

Preamble to Design Verification

Documentation

The division of the Eurocodes into numerous parts, each representing a separate subject, results in the requirement to refer to at least 14 Parts, possibly as many as 20 Parts, for the design of a composite highway bridge. In addition to EN 1990, for the basis of design, reference is needed at least to Eurocode 1 (Parts 1-1, 1-5, 1-6 and 2), Eurocode 2 (Parts -1- and 2), Eurocode 3 (Parts 1-1, 1-5, 1-8, 1-9, 1-10 and 2) and Eurocode 4 (Part 2). Foundation design will require reference to Eurocode 7. Other Parts will be needed for more complex structures, such as long-span cable-stayed bridges.

For structures in the UK, the UK National Annexes must also be consulted. Some of these National Annexes refer to BSI's 'published documents' (PDs), which are one form of "non-contradictory complementary information" (NCCI). NCCI has no special status according to the Eurocodes, it's merely what it says it is: a text book may well be considered as NCCI; industry is free to produce documents that may be so termed NCCI (as long as they do not contradict).

Design basis
The categorisation of design situations and combinations of actions to be considered is well set out in EN 1990 and is readily applied for ordinary highway bridges. The terminology used is clear and the use of 'Ed' and 'Rd' as subscripts for design effects and design resistances contributes to clarity in many aspects of the verification procedure. One minor oversight in the UK NA to BS EN 1990[3] is the omission of values for partial factors during transient situations (notably construction), although the use of factors for persistent situations should be conservative.

Material properties
Properties of steel and concrete material properties are clearly defined in EN 1993-1-1 and EN 1992-1-1 respectively. For concrete, the determination of long-term modulus of elasticity and shrinkage depends on the project-specific parameters of relative humidity and age at loading. No doubt, a common practice will evolve on 'normal' assumptions for these parameters at opening to traffic and later in the working life. In the examples, where shrinkage effects are favourable they are neglected, rather than using the lesser values at the time of opening to traffic.

The use of different coefficients of expansion for uniform temperature change and for temperature gradient will be puzzling to many but is easily dealt with.

Design Effects

Actions
Values for the density of steel and concrete are given in EN 1991-1-1. It is reasonable to use the lower end of the range for steel self-weight (77 kN/m^3); the suggested addition of 1 kN/m^3 for the self weight of reinforcement may be rather low for typical reinforcement in deck slabs and the designer should adopt a suitable value. For surfacing, the allowance for an additional thickness of 55% of nominal value given by the UK NA to BS EN 1991-1-1, **Table NA.1** is more onerous than the values given by BS 5400.

Strictly, the traffic load UDL (in Load Model 1, see EN 1991-2:2003, **4.3.2**) should only be applied to adverse areas of influence surface and these do not necessarily align with boundaries between traffic lanes. A more practical approach of applying the UDL over full lane widths, rather than part widths, will make very little difference to design effects. The use of a fixed width of notional lane (rather than dividing the available width into an integral number of lanes) and 'remainder' widths that are also loaded may seem a little odd but should cause little difficulty in practice. Thanks to the UK NA, the UDL has the same intensity over the full adverse area. Since the UDL does not vary with loaded length, it is simpler to apply than BS 5400 HA loading.

The single vehicle fatigue load model defined in EN 1991-2:2003, **4.6.4** is used in the simplified assessment of fatigue in EN 1993-2:2006, **9.2.2**. Although EN 1993-2:2003, **9.5.2** implies that a spectrum of lorry weights is needed for the simple assessment, no appropriate spectrum is offered in EN 1991-2. The UK NA to BS EN 1993-2 avoids the need for defining a suitable spectrum by giving an 'average' weight for UK traffic (the average is independent of the type of road). It is not entirely clear whether the fatigue load model should be applied in the same notional lanes as for Load Model 1 or in the marked lanes (as BS 5400 required). The reference in the UK NA to numbers of vehicles in 'slow' and 'fast' lanes (which are not defined) would indicate the use of the marked lanes. The use of notional lanes would be conservative.

Global Analysis

According to both Eurocodes 3 and 4 elastic global analysis is to be used for bridge structures, although the UK NA to BS EN 1993-2:2006, **NA.2.16** does allow particular projects to specify where plastic analysis would be acceptable (such as for accidental situations). No guidance is available on where plastic analysis might be appropriate.

In the elastic global models, both EN 1993-2 and EN 1994-2 require the use of effective widths of wide flanges, allowing for shear lag. Simplified approaches are given. However, if FE models are used, with shell elements for the deck slab, shear lag will be automatically taken into account and no allowance should be made in the model. However, this means that moments on notional composite beams extracted from such a 3D model should be determined from the stresses in the gross flange width, for verification against the resistance of a cross section that includes only the relevant effective width.

Traditionally, first-order (small deflection) analysis models have been used and buckling resistance has been determined by reference to empirical rules for 'effective length'. With the increasing power of modern software, elastic buckling analysis of a 3D model is now possible and this offers advantages in some situations. However, the interpretation of output from such software does require experience to ensure that appropriate buckling modes have been identified.

The models used in the Examples were 3D FE models (first-order) using shell elements for the deck slab and beam elements for the flanges, stiffeners and bracing members. An illustration of the ladder deck model, for a construction stage with only part of the deck slab, is shown in Figure 3.

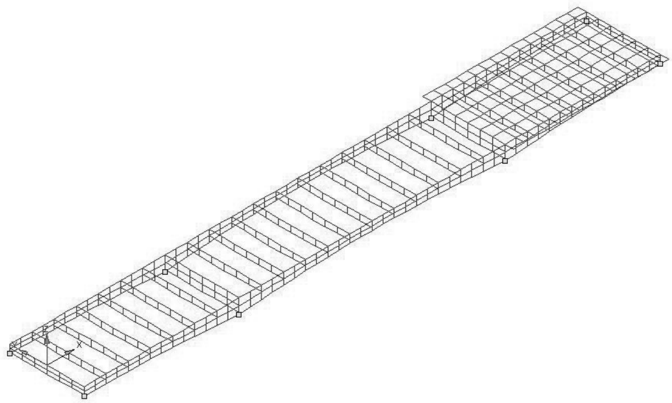

Figure 3. Global analysis model for ladder deck bridge (construction stage)

Design Verification of Beams

Resistance of cross-sections

Although Eurocodes 3 and 4 consider 4 classes of cross section, compared to the 'compact' and 'non-compact' designations in BS 5400, there is no significant change, since Classes 1 and 2 are treated in the same way for bridges and correspond to 'compact' sections. Class 4 is equivalent to a non-compact section with a web that is not fully effective (although Eurocode 3 takes a slightly less conservative view than BS 5400-3 of the loss of effectiveness).

When considering the design resistance of composite sections, care must be taken with the value of the design strength of concrete – the Eurocode 2 definition includes the parameter α_{cc} which, according to the UK NA has a value of 0.85; the design value is thus based on 85% of the cylinder strength whereas the Eurocode 4 definition bases it on 100% of the cylinder strength (but only mobilizes 85% of that strength on the compression side of the plastic neutral axis).

Shear resistance (of a beam web) is considered to be a property of the cross-section but it does depend on the stiffening along the beam. Where shear buckling resistance is mobilized, the partial factor γ_{M1} must be applied, in contrast to the γ_{M0} factor applied for design resistance of the cross section in bending. (The values of the factors are γ_{M0} = 1.0 and γ_{M1} = 1.1, according to the UK NA to BS EN 1993-2: 2005.)

Bending/shear interaction is only considered in relation to the bending resistance of the cross-section, not in relation to the buckling resistance of the member (as in BS 5400). Generally the limiting envelope is similar to that in BS 5400 except that is curved, rather than polygonal (see Figure 4).

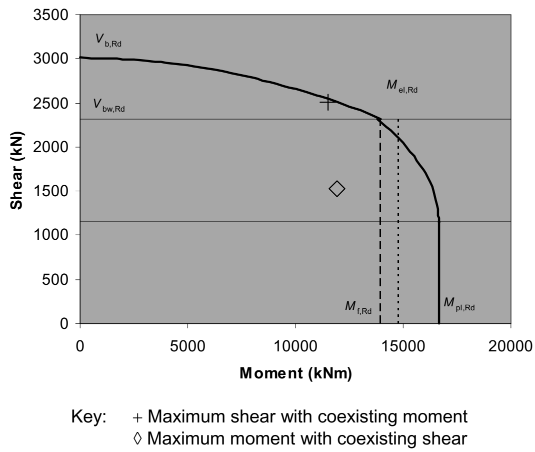

Figure 4. Limiting envelope for moment-shear interaction (Example 1)

One aspect not mentioned in the Eurocodes is the resistance of non-parallel sections, such as with a haunched profile or a curved soffit. In such sections some of the shear is carried by a component of the force in the inclined flange. It seems reasonable to continue the practice of reducing the design value of shear on a cross section (V_{Ed} in Eurocode terminology) by the vertical component of force in the compression flange. Slenderness of the web in shear should be based on the deeper end of a tapered web panel.

Buckling resistance of beams

For composite highway bridges, the two regions where buckling resistance needs to be verified are midspan regions during construction (at the wet concrete stage) and adjacent to intermediate supports (at the in-service stage) – the hogging moment regions. The first of these involves 'true' lateral torsional buckling (albeit with flexible intermediate torsional restraints provided by bracing or cross girders). The second is actually lateral distortional buckling, though EN 1994-2:2005, **6.4.2** says that the resistance for this mode of buckling may be evaluated using the reduction factor for LTB given in EN 1993-1-1 for the appropriate non-dimensional slenderness and buckling curve.

At the wet concrete stage, beams in multi-girder bridges are usually paired together and the LTB buckling mode involves the pair of beams twisting and displacing as a unit. Similarly, the main girders in a ladder deck are paired together by the cross girders. The mode and critical buckling load are influenced by the stiffness and spacing of bracing, and by the variation in bending moments and section properties across the span. The most accurate means to determine the elastic critical load, and thence the 'non-dimensional slenderness' and reduction factor for LTB that is to be applied to the resistance of the cross section, is by a 3D elastic buckling analysis. However, such analysis is not always available and even when it is, interpretation of output requires care and experience. One alternative is to adopt the empirical rules from BS 5400-3, which determine first an 'effective length' for buckling and from that the non-dimensional slenderness. The rules are available in SCI publication P356[1] and in PD 6695-2[4]. The Examples in P357 used the empirical rules and Figure 5 shows the deflected form of the ladder deck model when determining the torsional stiffness of the beams in the central span, as the first step to determining slenderness.

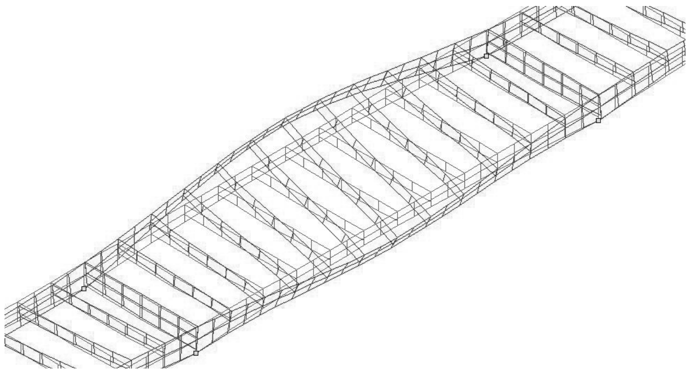

Figure 5. Displaced shape of bare steel ladder deck due to 'unit' moments about the longitudinal axes applied to both main beams over the central span

For hogging moment regions adjacent to supports, the elastic critical load for buckling of the bottom flange can also be determined using a 3D elastic buckling analysis and the beam verified using the general method of EN 1993-2:2006, **6.3.4.1**. However, EN 1993-2:2006, **6.3.4.2** offers a very useful simplified 'strut model' method. The flange plus part of the web is treated as a T-shaped strut and its lateral slenderness evaluated, making allowance for non-uniform force (along its length) and for flexible lateral restraint from the web. This is in effect a type of beam on elastic foundation model, as employed in the design of U-frames in BS 5400-3. In Example 1, the cross bracing (at about the quarter point of the span) was sufficiently stiff to be an effective lateral restraint and the slenderness was determined for the T-shaped flange plus part of the web, as a strut over the distance from support to bracing. In the ladder deck model the U-frame created by the first cross girder from the support and the web stiffeners to which it is attached was not sufficiently stiff to be an effective restraint to a strut over the distance from the support. The stiffness requirement at the second frame (which is less onerous because the strut is twice as long) was satisfied by the U-frame at that position and the slenderness was calculated for this longer length. The bottom flange in this case is curved and the depth of the beam decreases over the buckling length; this does not affect the determination of slenderness (the strut buckles laterally) but does require careful consideration of the values of moment and resistance at appropriate cross sections (both vary along the beam).

Restraint of deck slabs in ladder deck bridges

In the span regions of ladder deck bridges, the deck slab is effectively a wide plate in compression that is restrained against buckling, out its plane, by the cross girders. The cross girders thus need to be stiff and strong enough to provide that restraint; there are no explicit rules in the Eurocodes for the requirement but it is possible to give guidance that is compatible with other rules.

For the usual spacing of cross girders (about 3.5 m centres) the slab itself is sufficiently slender that second order effects in the slab would need to be allowed for, in accordance with EN 1992-1-1, although that verification is not given in P357.

Longitudinal shear connection

The strength of shear stud connectors to the new European Standard (EN ISO 13918) is about 10% less than the old BS 5400 studs but the design value of their shear resistance at ULS is comparable. As SLS, the limiting value of shear force per connector is 75% of the ULS shear resistance but since the design effects are typically no more than about 75% of the ULS values, SLS does not often govern – the main exception would be at the ends of the bridge, where primary forces due to temperature difference have to be transferred over a short length of girder).

In a 3D analysis, the proper 3D behaviour of the structure as a whole is modelled and thus, with loading that is not uniform across the width of the bridge, the notional composite beams (steel girder plus part of the deck slab) carry both bending and axial forces. The axial forces vary along the span and this variation may add to the longitudinal shear between the girder and the slab; in the Examples, the variation of axial force was taken into account, although it was modest in magnitude.

Design of Connections

Bolted connections

Apart from a change of terminology (from ‘HSFG bolts acting in friction’ to ‘slip-resistance shear connections’) the rules in Eurocode 3 are similar to those in BS 5400. Bolts for preloading are now supplied to EN 14399, usually in size M24 or M30. ‘Ordinary’ (hexagon head) bolts are available in Grades 8.8 and 10.9; type HRC (the generic designation for TCB)) is available in Grade 10.9.

Example 1 includes the design of a typical bolted splice. The flange and web cover plate connections were designed to resist the maximum forces at the splice position. The axial force in the top flange was found to be greatest, and compressive, at the wet concrete stage are consequently was magnified to allow for second order effects due to lateral buckling of the flange.

Example 2 includes the design of the lapped connection at the ends of cross girders. The connection detail and the ‘model’ of a width of concrete slab plus a steel web are shown in Figure 6. The most onerous requirements on this connection are adjacent to the intermediate support, where U-frame forces are developed due to loading on the cross girder and due to restraint of the compression flange. Determination of the design force on the extreme bolt depends on judgement of effective width of slab and thus the centre of rotation for the bolt group; in the absence of appropriate rules for effective width, a pragmatic judgement was made and the connection was designed against slip at ULS.

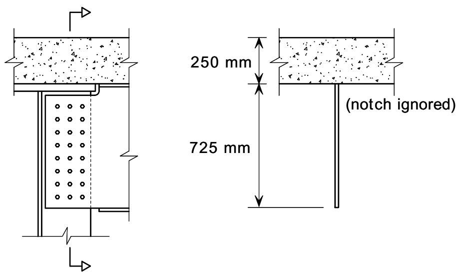

Figure 6. Main girder / cross girder connection in ladder deck bridge

Welded connections

The Examples verify the adequacy of the web/flange welds and the welds at the bottom of the bearing stiffeners, for static and fatigue loading. Weld sizes are expressed as throat thickness, which is consistent with the detailed rules in EN 1993-1-8, rather than leg length. When specifying weld size on drawings or other contract documents, it must be made clear when throat size is give, rather than the traditional UK practice of specifying leg length.

Conclusion

Preparation of the two worked examples revealed no major differences in design procedures (from those to BS 5400). Terminology is slightly different (and rather more precise) and procedures are sometimes a little more exacting. One omission from the Eurocode rules - the absence of simple expressions to calculate elastic critical moment, rather than resorting to a buckling analysis - is catered for either by use of its ‘simplified strut model’ or by use of NCCI adapted from rules in BS 5400. There is no indication that the Eurocode rules require a heavier structure than do the BS 5400 rules.

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OVERVIEW OF TIMBER BRIDGE DESIGN AND THE UK NA FOR EN 1995-2

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Abstract
Eurocode 5 'Design of Timber Structures' is a significant advance for the design of timber structures as it the first time that a limit state design code has been used in the United Kingdom. It also includes bridges within is scope, again a first for the UK. The change to limit state and the fact that a number of material standards have to be consulted to find the characteristic strength and stiffness properties mean that designers will take time to become familiar with the new standards particularly if they are not familiar with timber as a material. The first part of this paper discusses a number of these issues. One of the commonly expressed concerns about timber bridges is their durability. The second part of this paper reviews the recent work in Europe on this matter.

Notation

k_{mod}	Modification factor for duration of load and moisture content
γ_{m}	Partial factor for material properties, also accounting for model uncertainties and dimensional variations
R_{d}	Design value of a load-carrying capacity
$u_{\text{fin,G}}$	Final deformation for a permanent action G
$u_{\text{inst,G}}$	Instantaneous deformation for a permanent action G
k_{def}	Deformation factor

Introduction
Traditionally timber bridges in the UK have been small scale, typically those carrying pedestrians. However there have been some noted exceptions, for instance the timber viaducts in the West Country designed by Brunel to carry the South Devon and various Cornish railways. The last of these viaducts survived until the mid 1920s. Currently the main structural elements of most timber bridges in the UK are generally constructed using glued laminated timber or mechanically laminated beams with some having timber trusses as the main elements.

The first part of this paper will consider the requirements of Eurocode 5 with respect to the use of timber in bridges, a subject which was not covered by any of the British Codes of Practice. In particular it will provide references to the many additional documents that will be required to successfully design a timber structure. The second part of the paper will consider in more detail the important subject of durability and will use examples from around the world to demonstrate how durable timber bridges can be designed and constructed.

Eurocode 5 “Design of timber structures” is published in three parts:

- Part 1-1: General – Common rules and rules for buildings[1]
- Part 1-2: General – Structural fire design[2]
- Part 2: Bridges[3]

The main parts that will be of interest to the bridge engineer are Part 1-1 and Part 2. The latter part is a fairly small document which deals with a few bridge specific topics. It also gives details of limits on the applicability of Part 1-1 when considering bridges. Eurocode 5 is fairly ‘purist’ in that it does not contain material properties. These will be found in various material standards, as will be discussed later in this paper. Unlike Eurocodes 2 and 3 Eurocode 5 also contains a section (10) on workmanship.

All three of the National Annexes[4], [5], [6] are fairly small documents because the number of nationally determined parameters is small compared with some of the other Eurocodes.

Timber as a Material

Timber is a natural material that will inevitably show variability in its properties. Consequently when it is harvested and processed to form the raw material that is used in the construction industry it will require selection so that it is suitable for its intended use. This selection process is called grading and can either be done by machine or by visual methods. In the case of machine grading the stiffness or density of the timber is tested. A visually graded timber will be selected by the size of the knots, the angle of the grain and the width of the growth rings. For a particular species all of the above properties are indicators of the strength of the material.

Timber is a hygroscopic material consequently its moisture content will vary in relation to the humidity of the surrounding air. This will particularly apply to timber that is out in the open and needs to be allowed for in the design of the structure. A number of effects have to be allowed for. Firstly the strength and stiffness of a piece of timber will vary inversely with its moisture content. Over practical moisture ranges the change in strength will be in the range of 2% to 4% per 1% change in moisture content. Secondly the cross sectional size of the timber will vary with the moisture content. The amount of movement will depend on the species and whether the movement is in the radial or tangential direction relative to the grain. Below about 26% moisture content a typical softwood will move by about 0.25% perpendicular to the grain per 1% change in moisture content. Along the grain timber will hardly move with changes in moisture content. In order to make a practical system a number of service classes have been defined, see BS EN 1995-1-1: 2004 + A1:2008 **2.3.1.3** and **Table NA.2**. Parts of a bridge structure that are protected from the weather will be in service class 2 whilst those parts that are exposed directly to the weather will be in service class 3.

The strength of timber is also dependent on the time for which the timber is loaded. A timber that is loaded for say a couple of seconds will carry a much higher load than one that is loaded for say 50 years. A number of load duration classes have been defined, see Table 1 below. The duration of loading and the effects of moisture are allowed for by the use of the k_{mod} factor which can be found in BS EN 1995-1-1: 2004 + A1:2008 **Table 3.1**.

The durability of timber against fungal and insect attack depends on the species. Documents such as Wood Information Sheet 2/3-10[7] are a good starting point when considering what species of timber to use. Hardwoods vary from not durable to very durable, whilst softwoods vary from not durable to durable. Whilst preservative treatment can be used to increase the durability of a structure consideration of the design details is just as important as will be discussed later in this paper.

Whilst the clear felling of tropical hardwood forests has rightly created adverse publicity, the growth of a tree from infancy to maturity will absorb a lot of carbon dioxide and the production of timber from a well managed forest is good in environmental terms. There are a number of certification schemes which can be used, with care, to demonstrate that the timber has come from a sustainable source. Additionally the low amount of energy used to process and transport timber means that it is one of the more sustainable construction materials.

Interface with Eurocode 1990 'Basis of Design' and Eurocode 1991-2 'Traffic Loads on Bridges'

As discussed in previous papers the action combination rules are given in BS EN 1990 Annex A2 and its associated National Annex. However, when considering timber structures it is important to consider the duration of the applied loading. Strictly the worst combination has to be found for each load duration class as a different k_{mod} will need to be applied. In Table 1, below information from NA to BS EN 1995-1-1:2004 + A1:2008 **Table NA.1**, NA to BS EN 1995-2:2004 **NA.2.1** and NA to BS EN 1990:2002 + A1:2005[8] **Table NA.A2.4(B)** is combined.

Load duration class	Duration	k_{mod} for solid timber or glulam assuming service class = 3	Examples of loading	Partial factors for actions
Permanent	More than 10 years	0.50	Self weight	1.35
			Super-imposed dead	1.20
			Road surfacing	1.20
Long term	6 months to 10 years	0.55		
Medium term	1 week to 6 months	0.65		
Short term	Less than 1 week	0.70	Road traffic actions (gr1, gr2, gr5, gr6)	1.35
			Pedestrian actions (gr3, gr4)	1.35
			Snow (generally not needed)	1.50
Instantaneous		0.90	Wind actions	1.55 (50 year)

Table 1. Allocation of loading to load duration class and values of k_{mod}

This table shows that long term and medium term load durations will not need to be considered. Assuming that a partial factor of 1.35 is applied to all the permanent loads it can be shown that for a structure in service class 3 conditions the short term load case is the critical load duration class if the short term actions are more than 40% of the permanent actions. This will typically be the case unless a concrete deck is used.

Materials and Partial Factors

Eurocode 5 allows the use of a wide range of timber materials and timber derived panels. In practice durability requirements, as discussed later, will limit the choice of materials that can be used. Unfortunately the basic strength and stiffness properties are not given in Eurocode 5 but are given in one of the material standards. This is a change from BS 5268-2[9] which significantly reduces the usability of the Eurocode for those that do not use timber on a regular basis.

Solid timber

There is a wide range of solid timber products available in the UK, however, only timber that has been graded in accordance with BS EN 14081-1[10] should be used structurally in bridges. BS EN 14081 covers material that has been machine graded directly to a strength class and acts as a 'head' standard for national visual grading standards. In the UK the visual grading standards are BS 4978[11] and BS 5756[12]] for softwoods and hardwoods respectively. Reference will need to be made to BS EN 1912[13] to convert these visual grades to strength classes. Once the strength class is known then the basic strength and stiffness properties can be looked up in BS EN 338[14].

The material partial factor for solid timber ($\gamma_M = 1.3$) is given in the UK National Annex to BS EN 1995-2[6] **Table NA.1**.

Glued laminated timber

Glued laminated timber, often abbreviated to glulam, should be manufactured in accordance with BS EN 14080[15] with the strength and stiffness properties being given in BS EN 1194[16]. This however could soon change as EN 14080 is currently being revised and a number of standards, including EN 1194, may be absorbed into EN 14080.

BS EN 14080 covers glued laminated timber manufactured using coniferous (i.e. softwood) timber species and gives two types of grades of glulam. If the glulam is manufactured from the same grade of timber throughout its depth then it will be a homogenous glulam, e.g. GL 28h, whilst if lower quality timbers are used in the middle of the beam then it will be categorised as a combined glulam, e.g GL28c. Combined grade glulam is optimised for use in bending however it should be remembered that the timber in the middle zone will have a lower density and this will need to be considered when designing connections. The reference size for glulam is 600mm and the grade bending and tensile stresses may be increased if the depth or width, respectively, is less than 600mm.

The material partial factor for glulam ($\gamma_M = 1.25$) is given in the UK National Annex to BS EN 1995-2[6] **Table NA.1**.

Plywood

If plywood is used on bridges it will need to comply with BS EN 636[17], the class of plywood depending on the exposure. The appropriate class is required to ensure that the adhesive has adequate moisture resistance. It is however essential to check that the timber from which the plywood has been manufactured has adequate durability for the intended situation. It can be very difficult to treat plywood with preservatives post manufacture, because the gluelines inhibit penetration of the preservative. Where the plywood is at risk of decay, it is therefore necessary to use a plywood made from timber of adequate natural durability. In practice the only such plywood currently available, traditionally referred to as marine ply, is not made from structurally graded veneers and will therefore be limited to non-loadbearing uses such as wearing surfaces. It will also be important to ensure that the edges are protected from standing water, otherwise the end grain will rapidly soak up water and delaminate. For instance plywood can be used to provide a slip resistant surface by coating the top with gritted epoxy, but the edges should also be given an epoxy coating so that any trapped water cannot be absorbed into the edge of the ply.

Deck Systems and Distribution of Concentrated Vertical Loads

BS EN 1995-2 has a number of deck systems which will not be familiar to designers in the UK. For footbridges the typical UK detail is to use grooved planks with small spaces between so that water does not collect on the surface. This detail has the disadvantage that point loads are not distributed between the boards. In the case of pedestrian loads, both the UDL and the point load, this is not of concern. However if service vehicles need to be allowed for the loads under the wheels of these vehicles will become the ruling criteria. The Eurocode contains design rules for a number of laminated deck plate systems, including laminated deck plates, stress-laminated deck plates or cross laminated deck plates, see Figure 1. The advantage of these systems is that patch loads will be distributed better but allowance needs to be made in the design for the dispersal of water from the deck.

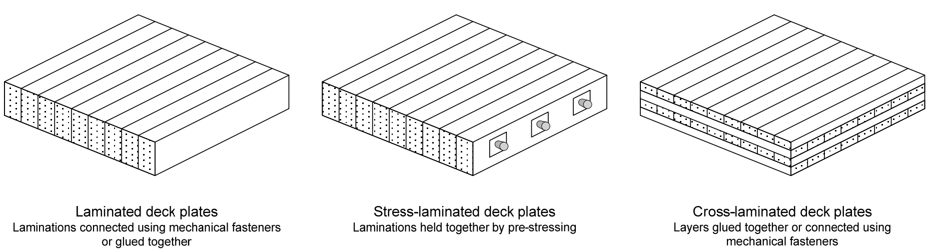

Figure 1: Laminated deck systems

BS EN 1995-2 also contains some guidance on the use of concrete decks acting compositely with the timber structure. These have a number of advantages for vehicular decks in that a wearing course can be applied to the top of the concrete and can be replaced using normal highway techniques when it has reached the end of its life. However many UK engineers with timber experience would not look favourably at the use of a heavy weight deck on a light weight structure.

Generally concentrated loads are dispersed at an angle of 45° from the surface down to the mid height of the timber structural element. However in the case of laminated deck plates, except for cross laminated plates, the dispersion angle is only 15° when perpendicular to the grain, to reflect the weakness of the structure in that direction.

Ultimate Limit State

Timber is a brittle material and therefore timber structures are designed assuming elastic, rather than plastic, bending theory. However it must be remembered that the shortest duration load for the particular load combination and the moisture content must be taken into account when calculating the design resistance, that is:

$$R_d = k_{\text{mod}} \frac{R_k}{\gamma_M} \tag{1}$$

As k_{mod} is generally less than one there is a concern within the timber industry that this factor may be forgotten leading to an unsafe design because in BS 5268-2[9] it was generally conservative if a modification factor was missed.

If the lateral stability of any part of the structure is provided by the deck system then the effects of movement in the deck due to moisture changes will need to be considered. It may be necessary to consider the replacement of deck elements due to the abrasion of the wearing surface. BS EN 1995-2 **Section 6** contains guidance on the ultimate limit state design of deck plate systems and states that fatigue should be considered, except in the case of footbridges. Reference is made to a simplified verification method in BS EN 1995-2 **Annex A**.

Serviceability Limit States

When checking the serviceability limit state it will be necessary to consider both the long term deflection, including creep, under permanent loads and the additional short term deflection under variable loads, see BS EN 1995-1-1 **2.2.3**. The long term deflection under permanent loads is given by:

$$u_{fin,G} = u_{inst,G}(1 + k_{def}) \tag{2}$$

In the case of solid timber or glulam in service class 3, $k_{\text{def}} = 2.0$, so that the long term deflection is three times the instantaneous deflection calculated using the mean moduli of elasticity and shear moduli. As $\psi_2 = 0$ for variable loads relating to traffic there is no long term component of load and therefore only the short term deflection needs to be considered. The chosen limiting values in NA to BS EN 1995-2 **Table NA.2** are the recommended values.

Whilst BS EN 1995-2 **7.3** and **Annex B** give some guidance on vibrations caused by pedestrians design guidance elsewhere in the industry should also be consulted.

Connections

The main guidance on the use of connections can be found in BS EN 1995-1-1: 2004, **Section 8**. However BS EN 1995-2: 2004, **8.1(1)P** states that axially loaded nails, stapled connections or punched metal plate fasteners should not be used in bridges. In practice most structural connections in bridges will be the larger bolts and dowels with some use of connectors like shear plates and split rings.

The presentation is very different to that in BS 5268-2[9] in that no tables of design values are given. Instead, for lateral load carrying dowel type connections numerous equations are presented, see BS EN 1995-1-1, **8.2.2** and **8.2.3**, that have been derived from work by Johansen[18] which assumes that both the fastener and timber are ideal rigid-plastic materials. It is also necessary to comply with the spacing rules in BS EN 1995-1-1 **Section 8** to ensure that splitting of the timber perpendicular to the grain is avoided. The number and complexity of the expressions in the Eurocode means that in practice spreadsheets or a specially written computer program are likely to be needed for calculating the load capacities of connections.

It is essential to remember that most of the expressions in BS EN 1995-1-1: 2004, **Section 8** are used to calculate the characteristic strength, as indicated by the 'Rk' suffix and will need to be divided by an appropriate material factor of safety, γ_M, which is 1.3 in the case of strength verification and multiplied by the appropriate k_{mod} to determine the design value.

BS EN 1995-2 **8.2** gives guidance on timber-concrete connections for composite timber-concrete composite structures. This is an area where there is very little experience in the UK and it will be necessary to look at experience elsewhere in the world when designing and detailing these connections.

Durability

Achieving an adequate life is the single most important factor in the design of a timber bridge. The approach adopted will govern both the choice of structural form and the overall aesthetics of the bridge.

The mechanisms of timber decay

Timber is at risk of attack by both fungi and insects. In practice insects are not generally a problem in cooler climates such as the UK and fungi require a minimum timber moisture content of 20% to germinate. Protected structures, outside but under cover, normally dry down to 12-16% moisture content, well below the threshold for decay. However, bridges are directly exposed to the rain and therefore at potential risk of fungal attack if the wood remains above 20% moisture content for long periods. Any exposed end grain is particularly vulnerable because it quickly absorbs water. The other risk areas are surfaces in close contact (where water can become trapped by capillary action), timber in contact with the damp ground, water traps and any horizontal surfaces where water can collect allowing time for it to soak into the timber particularly through any drying fissures.

If timber does remain wet for long periods the only way to prevent fungal attack is by injecting toxins into the wood, creosote and CCA (copper chrome arsenic) being the traditional candidates. The heartwood of certain timbers, such as European oak and ekki, also contain natural toxins which impart varying degrees of resistance to decay.

Traditionally, timber bridges were covered. The principle was simple. The main structural members, by definition difficult and expensive to replace, were protected from the rain by a roof and walls. The roof and walls were usually also in timber, but could be easily replaced as and when required. Most importantly, when they failed, the leaks were visible, and even if the leaks were allowed to continue, the inside of the bridges were well ventilated, promoting quick drying. (Remember that the timber must remain continuously wet for decay to continue). Where available, oak was used for the main members, which with its natural toxins, provided further protection against decay due to failure of the roof and cladding. Additional details included lifting the timber up on stone plinths, to guard against moisture from the ground.

Modern approaches to achieving durability

The use of naturally durable timbers

While several hardwoods are naturally durable, the same extractives that make them durable often make them difficult to glue. Such bridges are therefore generally either bolt laminated (a method typically used for arches, particularly in ekki, see Figure 2) or trussed built up from solid baulks (both Ekki and European oak are often used in this way). Larch is sometimes used; being a softwood it can be readily glue laminated, but is only moderately durable and therefore would need to be combined with protection details to keep the timber dry in order to achieve an adequate life.

Figure 2: Bolt-laminated Ekki bridge at Finowfurt, Germany.

The use of preservatives

Preservative treatment is covered in BS 8417[19], which follows the EN system.

There are three requirements for effective preservative treatment: (a) the preservative needs to be applied under pressure in a vacuum tank; (b) the timber needs to be sufficiently dry; and

(c) a sufficiently permeable species needs to be used (typically Scots pine in Europe). If these requirements are met, more or less complete penetration of the sapwood can be achieved, if the treatment time is long enough. However, the heartwood (this being the material towards the centre of the trunk) is relatively resistant to the uptake of preservative and therefore remains at risk of decay, albeit protected by a small degree of natural durability. If treating a finished glulam, some sapwood in the centre of the member will also remain untreated since both the gluelines and the heartwood block penetration of the preservative.

There are two types of preservative suitable for full exterior exposure – heavy oil-borne and copper based waterborne.

The heavy oil-borne preservatives (traditionally creosote) tend to be the most effective. This is because they are highly toxic and because the oil acts as a moisture barrier, keeping the timber drier and reducing the risk of fissuring due to rapid surface drying which might allow water to reach the untreated heartwood. The use of such preservatives over many years suggests they can meet the 50 year design life required on minor roads in the US. Obviously the oily surface of the timber cannot be glued, so that treatment must be carried out after fabrication of the glulam and ideally after all cutting and drilling to ensure that the timber exposed at the surface is fully treated. The disadvantage with oil-borne treatments is that the oil is not chemically fixed to the timber and can therefore stain clothes or leach out (particularly in hot weather); it also carries a noticeable odour. There are also potential health and environmental risks although given the tiny amounts of oil which do leach out, the latter are probably more perceived than actual. Recent efforts have concentrated on retention levels and improving post-treatment cleaning.

The alternative, particularly for members in human contact (such as handrails and decking), is to use waterborne preservatives (traditionally CCA - copper chrome arsenic; now restricted and being replaced by copper combined with organic fungicides). Although the waterborne preservatives are still toxic (it is of course this toxicity which prevents the fungal growth), they have the advantage of being chemically fixed to the timber. Because the preservatives are waterborne, they swell up the timber leading to fissuring as the wood dries and shrinks; it is therefore most common to treat the individual laminates before gluing which at least ensures full penetration of the sapwood. Obviously without the moisture barrier provided by the oil-borne treatments, there still remains the risk of subsequent fissuring of the completed glulam in hot weather exposing the untreated heartwood. While there is little long term field data available on the efficacy of the modern waterborne formulations which have replaced CCA, the combination of the fissuring (discussed above) and reduced fixity and toxicity levels of the modern formulations (following restrictions on the use of chromium and arsenic respectively) suggests that they are likely to offer a shorter life than the traditional oil-borne treatments such as creosote. The BWPDA (the British Wood Preserving and Damproofing Association) currently recommends a 30 year design life for members with full external exposure.

To achieve a design life longer than about 50 years then it is also possible to use a double treatment process for glulam members – the laminates are pressure impregnated with a copper waterborne preservative before gluing and the completed glulam is then pressure impregnated with creosote. By treating the laminates before gluing, through thickness treatment of the sapwood can be achieved. The outer shell of creosote makes an effective water repellent (to help reduce swelling and subsequent drying fissures), as well as adding further decay resistance.

To achieve very long design lives, then for added protection, upper surfaces of the main members can be capped with copper, to prevent water entering any fissures, see Figure 3. The expensive capping details tend to be left off secondary members, such as plan bracing between arches, which can be easily replaced if required.

Figure 3. 70m span bridge at Tynset, Norway; one of the longest timber bridges in the world designed for full highway loading. To achieve the 100 year design life the glulam members were treated with CCA before laminating and creosote after laminating and the upper surfaces were also protected with copper caps.

Unfortunately, a preservative which is toxic to fungi will also by definition be toxic to man and other animals, meaning that reliance on preservatives is probably not a viable long term approach. Very small amounts of creosote will drip off the treated members and human contact with the surface of creosote treated timber is better avoided. CCA is now heavily restricted mainly because the treated timber was difficult to dispose of (for example the chrome and arsenic are driven off on burning). It is therefore interesting to look at the central European approach to timber bridge design.

The use of protection methods to keep the timber dry

In principle, the central European approach is very simple. Inspired by the traditional covered bridges, it aims to prevent decay of the spruce (the main European forestry timber, but a species with no inherent durability against decay and one that is resistant to preservative treatment) by keeping the timber dry, but with a local cover rather than an expensive roof. An early and probably the most famous example is the ribbon bridge at Essing, see Figure 4, completed just over twenty years ago. It represents the peak of timber engineering at the time and also shows how the methods of protection have developed based on subsequent experience.

Figure 4. The famous ribbon bridge at Essing relies on in-situ finger joints to form the continuous glulam beams, which are protected by the wooden deck.

From the start emphasis was placed on protecting the main structural members. These comprise full length glulam beams (very carefully finger jointed in-situ) supported on timber legs – both beams and legs are more or less impossible to replace. The top of the beams are protected from the rain by a metal roof, concealed below the timber deck, while the sides of the members are painted, which slows both the uptake of water and the loss of water (which can lead to fissuring due to rapid surface drying and shrinkage), as well as screening the surface of the timber from UV light (which can degrade the surface of the timber and debond the paint). However, even with more reflective white paint it is difficult to prevent rapid surface drying of the timber faces exposed to the sun, creating fissures some of which will slope downwards into the timber, providing a trap for rainwater, and leading to local decay. Thus, as with domestic timber windows unless the paint is reapplied as soon as any cracks occur, there is a risk of water ingress and decay. While windows can be locally filled, structural members are rather harder to repair. Timber/timber and timber/steel mating surfaces are also vulnerable because water gets drawn in by capillary action.

Visitors to the bridge today will see that a sympathetically detailed side cap has now been added, see Figure 5, with a ventilation gap behind, to provide complete protection from the rain. This prevents decay to the sides of the main members and prevents water becoming trapped under the nail plates that were fixed to the vertical faces of the beams. Surprisingly no

Figure 5. Detail of Essing showing the protective cladding that has recently been added to the sides of the ribbon beams.

Figure 6. In this bridge at Wernau, Germany, the untreated spruce glulam is protected from wetting by the overhanging precast deck.

protection has been added to the legs, although arguably these are less vulnerable since any fissures which do develop will tend to slope downwards (parallel to the grain) and therefore be fairly free draining. More recent designs have followed a similar approach, now embodied in the new German standard for timber bridges (DIN 1074[20]), in which the timber is fully protected top and sides either by an overhanging deck, see Figure 6, or with side cladding, see Figure 7. The aim is to guarantee a long life, with similar maintenance costs to steel and concrete bridges (typically 1% pa).

Figure 7. Metal caps and larch cladding are used to protect the main untreated spruce glulam members of this bridge near Aachen, Germany.

The cover is usually provided by the deck. However, rather like a flat roof it is difficult to ensure a completely watertight seal. Remembering that the timber is in a perishable species (spruce) and that by definition the upper surface immediately below the deck will be poorly ventilated, the detail is very vulnerable. It is for this reason that more robust details are now being developed [21], although these have yet to achieve codified status. These details accept that the waterproofing will never be perfect and therefore seek to provide a second layer of protection by providing falls to the upper surface of the timber as well as a ventilation cavity (see Figure 8). Most importantly, since any decay of the timber will occur in poorly ventilated and therefore invisible areas, it is vital that the secondary protection system is configured to ensure that any leaks in the primary barrier will be visible by, e.g. providing sumps and vertical drainage pipes in the upper surface of the timber, as well as leaving inspection holes to allow for future inspection.

Figure 8. This bridge at Hochstetten, Germany improves on the details at Wernau – as shown in the detail the top of the timber is laid to falls and a sufficiently large ventilation cavity is provided above to allow future inspection.

Design life and maintenance costs

Design life in Germany has yet to be fully codified, however current proposals are for a figure of 80 years where the main structural timbers are fully protected from wetting and a lower figure of 40 years for unprotected or partially protected members which rely mainly on pressure impregnation with a suitable preservative or an inherently durable timber.

80 years is credible for fully protected structures – if the timber is kept dry it should last indefinitely. Critical structural members, which would be difficult to replace and whose loss could lead to safety issues, should be adequately protected from wetting, be that by a proper roof or local cladding combined with secondary protective measures such as adequate ventilation to allow any water ingress which does occur between periodic inspections to quickly evaporate.

40 years for unprotected structures is probably optimistic unless they rely on heavy oil based preservatives, through-thickness treatment of the individual laminates with CCA before gluing, or the use of a very durable tropical hardwood such as ekki. Durable species such as European oak (particularly cost effective if used green) or moderately durable species such as larch can also be used, but to try and achieve a 40 year life would require very careful detailing to avoid water traps and to protect horizontal surfaces and end grain. However, the Achilles heel remains the risk of water becoming trapped in surface fissures – either drying fissures of the green oak or (particularly in larch glulam) due to the large differential shrinkage and swelling of the laminates with varying moisture content. Thus the naturally durable timbers are possibly more useful materials for the fully exposed parapets and decking, rather than for the main structural members.

However, designers cannot sensibly discuss design life without providing guidance on the required levels of inspection and maintenance. Ideally, the maintenance will consist of cheap routine measures (e.g. replacement of cladding, maintenance of rainwater goods etc) to ensure that the main structure, which is at best expensive to replace, remains undamaged. Inspection during wet weather will help show the effectiveness of drainage measures, drip details etc so that they can be improved before problems occur.

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OVERVIEW OF GEOTECHNICAL DESIGN OF BRIDGES AND THE PROVISIONS OF UK NA FOR EN 1997-1

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Abstract

This paper provides an overview of aspects of BS EN 1997-1:2004 relevant to bridge design, highlighting some key changes from past bridge design practice in accordance with BS5400[1] and the Design Manual for Roads and Bridges (DMRB)[2]. In addition, it describes the background to provisions of the UK National Annex (NA) for BS EN 1997-1:2004 particularly relevant to bridge design.

Introduction

BS EN1997-1:2004 sets out principles and requirements for the geotechnical aspects of the design of buildings and civil engineering works. The use of BS EN1997-1:2004 in bridge design brings some quite significant changes from past UK practice.

The former British Standard used in bridge design, BS5400[1], and the Standards used for associated aspects of geotechnical design were not consistent in their limit state philosophies, and this could lead to complications when they were applied in conjunction with one another. It is therefore a significant step forward that BS EN1997-1:2004 is included within the suite of Eurocodes, and that it follows the limit state philosophy established in BS EN 1990:2002. It is, however, this alignment of limit state philosophies between structural and geotechnical design and the associated treatment of soil-structure interaction that gives rise to some of the more significant impacts on related aspects of bridge design.

This paper provides a brief overview of the aspects of BS EN 1997-1:2004 relevant to bridge design, highlighting some key changes from past bridge design practice in accordance with BS5400[1] and the DMRB[2]. It also describes the background to some of the provisions of the UK National Annex (NA) for BS EN 1997-1:2004 relevant for bridge design. It does not cover ground investigation or the evaluation of geotechnical parameters. More comprehensive background and guidance on the application of BS EN1997-1:2004 is provided by Bond and Harris[3] and Frank et al[4].

The partial factors for actions used in bridge design and some of the requirements relating to the way that actions are combined are different from those for buildings (see BS EN1990:2002 **Annex A1** and **Annex A2** for buildings and bridges respectively). Some published guidance documents on the application of BS EN 1997-1:2004 use the provisions for buildings and do not highlight this distinction.

Overview of Geotechnical Design for Bridges

General

BS EN 1997-1:2004 was one of the most challenging Eurocode parts to develop. It has a strong emphasis on the principles that need to be satisfied in geotechnical design, but often provides limited coverage on the geotechnical models that should be used to satisfy these principles. It therefore offers flexibility to designers across Europe to select geotechnical models appropriate to the project and ground conditions.

Clearly, this flexibility demands knowledge and understanding from designers, and BS EN 1997-1:2004 therefore highlights in its assumptions that the data required for design should be ‘collected, recorded and interpreted by appropriately qualified personnel’ and that structures should be designed by ‘appropriately qualified and experienced personnel’.

British Standards for geotechnical design, such as BS 8002[5] and BS 8004[6], which are no longer current standards, typically provided greater coverage of geotechnical models than BS EN 1997-1:2004. As acknowledged in UK National Annex to BS EN 1991-2, **NA.4**, the non-conflicting parts of these documents remain valuable sources of complementary design information.

Limit state geotechnical design

Although limit state standards have been well established in structural design for many years, the same is not the case for geotechnical design standards. The development of limit state geotechnical design standards presents some particular challenges that do not generally apply to structural design standards.

Firstly, for structural standards it is possible to specify the material properties, structural detailing requirements and construction tolerances for which the analysis methods and resistance models are valid. Clearly in geotechnical design, it is not possible to apply such constraints. A geotechnical design standard needs to be generally applicable across a broad range of ground conditions that may be encountered in practice.

Secondly, in limit state structural structure design, partial factors are generally applied separately to actions (to determine design effects) and to material properties (to determine design resistances). The design is verified by comparing the design effects and the design resistances and there is no direct coupling of uncertainty in actions and materials. This is not typically the case even in the simplest geotechnical design problems. Fundamentally, this is because soil differs from structural materials in being reliant on friction for its strength.

Consider, for example, the case of determining the horizontal earth pressure applied to a retaining wall. The horizontal earth pressure is conventionally determined as the product of the vertical effective stress in the soil and the relevant earth pressure coefficient, K. This earth pressure coefficient will be dependent on the angle of shearing resistance of the soil, φ', amongst other parameters.

Thus it is clear, even in this simple case, that there is a coupling of the effects of the weight of the soil (affecting the vertical effective stress) and the soil material properties (affecting K). Expressed in more general terms, it can be seen that geotechnical actions can be dependent upon geotechnical material properties. Similarly, it is often the case that geotechnical resistances are dependent on geotechnical actions. This coupling adds complexity to the application and calibration of partial factors, as discussed further below.

BS EN 1997-1:2004 requires that designs are verified for ultimate and serviceability limit states (see BS EN 1997-1:2004, **2.4.7** and **2.4.8**). Ultimate limit states which involve failure or excessive deformation of the ground (GEO) are relevant to the design of bridge abutments and foundations include sliding and bearing failure. Serviceability limit states are primarily concerned with settlement, other displacements and cracking of structures.

As discussed further below, in the verification of sliding, partial factors are applied in a different fashion from the lumped factor of safety approach typically used in past bridge design practice (see e.g. BD 30/87[7]). In contrast to common past design practice for spread foundations in bridge design, which frequently was based on a (single) allowable bearing pressure calculation, BS EN 1997-1:2004 now requires separate verifications for (ULS) bearing resistance and (SLS) settlement of foundations (although see BS EN 1997-1:2004, **2.4.8(4)**).

Spread foundation settlement consideration in bridge design

In bridge design, it will usually be desirable for limits on settlements to be defined early in a project. These limits can then be applied as actions in the structural design, and the foundations can be designed so that these limits are not exceeded. BS EN 1990:2002, **A2.2.1 (13)-(17)** explains how the effects of differential settlements should be taken into account in the structural design.

BS EN 1990:2002 gives three different combinations of actions that are used for verifying serviceability limit states (see BS EN 1990:2002, **6.5.3** and Denton et al[8]), namely the characteristic, frequent and quasi-permanent combinations. Most of the Eurocode parts are explicit in stating which combination of actions should be used for specific serviceability limit state verifications.

BS EN 1997-1:2004 does not, however, state explicitly which combination of actions should be used for settlement calculations for spread foundations. It does state (clause **6.6.2(1)**) that calculations of settlements shall include both immediate and delayed settlement. BS EN 1990:2002 highlights, however, that settlements are mainly caused by permanent loads and backfill (clause **A2.2.1(15) NOTE 1**), although it goes on to state that variable actions may have to be taken into account for some individual projects.

The intention of BS EN 1997-1:2004 and BS EN 1990:2002 therefore appears to be clear, although it is not expressed using the terminology of BS EN 1990:2002, **6.5.3**. Often for bridge designs, settlements will be dominated by those that arise over time due to permanent effects, and therefore they can be evaluated using the quasi-permanent combination of actions with reasonable accuracy. However, designers will need to consider whether settlements will be significantly affected by occasional heavy loads (variable actions), in which case these actions and their duration of application will need to be taken into account.

There is an important note of caution here, and one that illustrates the need for clear communication between the structural and geotechnical engineer and a shared understanding of the Eurocode approach to limit states and the combination of actions. It has been common practice (if not necessarily good practice), for geotechnical engineers to provide bridge designers with values of allowable bearing pressure to be used in sizing foundations and for bridge designers to check that these allowable bearing pressures are not exceeded under the effects of both permanent actions (dead and superimposed dead loads) and traffic loads. If these same allowable bearing pressures are used for Eurocode designs for settlement verifications using the quasi-permanent combination of actions (based on 'comparable experience'), the resulting design could be significantly different with greater settlements resulting than in past designs. This is because the value of ψ_2 for traffic actions is zero, so the quasi-permanent combination does not include any traffic load effects.

Thus, whilst the geotechnical engineer may consider that 'comparable experience' gives confidence that settlements will be acceptably small, they may not realise that the treatment of loads used by the structural engineer would have changed quite considerably.

Geotechnical category

BS EN 1997-1:2004, **2.1** introduces the concept of geotechnical category as a basis for establishing geotechnical design requirements. Three categories are established reflecting the complexity of the structure and level risk, ranging from Category 1 for small and relatively simple structures with negligible risk up to Category 3 for the most complex and challenging projects. Most conventional bridge designs will be Category 2. This categorisation will be familiar to many bridge designers because it is incorporated in the DMRB Standard HD 22/08[9] which sets out requirements for managing geotechnical risk.

BS EN 1997-1:2004, **2.1(4)** states that limit states should be verified using either one, or a combination of, the use of calculations, adoption of prescriptive measures, experimental models and load tests, or, an observational method. It further states that designs for structures in Geotechnical Category 2 should normally include quantitative geotechnical data and analysis and routine procedures for field and laboratory testing (BS EN 1997-1:2004, **2.1(18)** and **(19)**). This is consistent with past UK design practice for bridges.

Design approaches

As discussed by Denton et al[8], BS EN1990:2002 gives three sets of partial factors for actions that are used in ultimate limit state verifications. These Nationally Determined Parameters (NDPs) are referred to as Sets A, B and C and are given in UK National Annex to BS EN1990:2002, Tables **NA.A2.4(A)**, **NA.A2.4(B)** and **NA.A2.4(C)** respectively. For persistent and transient design situations, Set A is used for verifications of static equilibrium (EQU) and Set B is used for verifications of structural resistance (STR) where there are no geotechnical actions or geotechnical resistance present.

For persistent and transient design situations where geotechnical actions or geotechnical resistance are present in a verification of structural resistance (STR) or in the verification of resistance of the ground (GEO), BS EN 1990:2002, **A2.3.1(5)** explains that one of three *design approaches* should be used. The choice of design approach is an NDP, and in the UK Design Approach 1 is to be used. The reason for the inclusion of three different design approaches stemmed from the difficulties in getting agreement on how partial factors should be applied, and in particular whether partial factors should be applied to ground properties or ground resistances. In Design Approach 1 both the Set B and Set C partial factors on actions are used.

It is BS EN 1997-1: 2004, **2.4.7.3.4** that explains the full application of the three design approaches (for ULS verification of STR/GEO for persistent and transient design situations). In Design Approach 1, with the exception of the design of axially loaded piles and anchors, partial factors greater than unity are applied to ground properties and partial factors equal to unity are applied to ground resistances. Two separate calculations are required using different sets of partial factors, referred to in BS EN1997-1:2004 as combination 1 and combination 2. (It is helpful to observe that the meaning of the words 'set' and 'combination' in this context is their conventional linguistic sense, and should not be confused with the specific meanings used in BS EN1990:2002 for 'Set A, B and C' partial factors and 'Combinations of actions').

The way in which partial factors are applied in combinations 1 and 2 is defined in BS EN1997-1:2004, **2.4.7.3.4.2**, and is illustrated in Figures 1a and 1b respectively. Unfortunately, as shown in Figure 1, BS EN1997-1:2004 and BS EN 1990:2002 use a different terminology for the partial factors applied to actions, with Sets A1 and A2 in the former meaning Sets B and C in the latter.

The way in which partial factors are applied in Design Approach 1 is demonstrated by Christie et al[10].

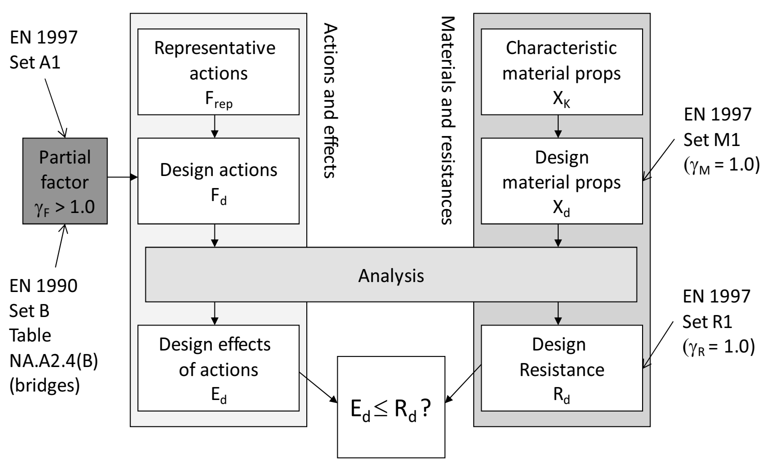

Figure 1a. Application of partial factors in Design Approach 1, Combination 1 (STR/GEO, persistent or transient design situations, except axially loaded piles and anchors), modified from Bond and Harris[3]

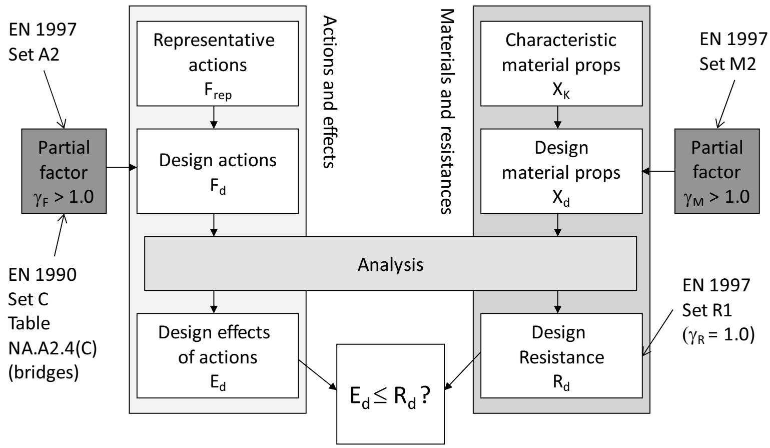

Figure 1b. Application of partial factors in Design Approach 1, Combination 2 (STR/GEO, persistent or transient design situations, except axially loaded piles and anchors), modified from Bond and Harris[3]

Provisions of UK National Annex for EN1997-1

The following sections provide background to values given in the UK National Annex to BS EN 1997-1:2004 relevant to bridge design. Background to the UK NDPs for piles is given by Bond and Simpson[11], [12] and is not included in this paper.

Decisions on the Status of Informative Annexes (NA.3)

Annex B (NA 3.1)

As discussed above, during the development of BS EN1997-1:2004 there was considerable debate about the appropriate way for partial factors to be applied in geotechnical design. An overall consensus could not be reached and as a result BS EN1997-1:2004 allows three different design approaches to be used for the STR/GEO limit state verifications. Annex B aims to clarify the background to these different design approaches. However, when initially applying BS EN 1997-1:2004 there can appear to remain some ambiguity in the way in which partial factors should be applied.

This ambiguity arises because BS EN1997-1:2004 does not always differentiate consistently between situations where no partial factor is applied to a parameter and cases where a partial factor is applied, but is equal to 1.0. For example, **Annex B** explains that, “in most cases Design Approach 1 adopts Equations (2.7a)”; this equation defines the design resistance and does not include a partial factor on resistance. However, clause **2.4.7.3.4.2(1)P** generally requires that partial factors for resistance from set R1 are used for Design Approach 1; accordingly recommended values are provided for Set R1 in **Annex A** and these have been retained in the National Annex. Because the values of the partial factors in Set R1 are all 1.0, Equation (2.7a) yields the same numerical result as the complete expression for resistance given in Equation (2.7c).

The guidance in the National Annex on the application of Annex B and also in National Annex clause **A.1** aims to address the potential for confusion.

Annex C (NA 3.2)

Annex C provides useful guidance on the evaluation of earth pressures. However, there were a few issues that merited some clarification, and the National Annex does so. Firstly, the note under BS EN1997-1:2004, **Equation (C.9)** states that the expression is on the safe side. Although not stated, this appears to assume that active pressure is unfavourable and passive pressure is favourable. However, this is not necessarily the case in bridge design, see discussion of National Annex clause **A.2.2** below.

Secondly, the values of K_a and K_p given in Figures **C.1.1** to **C.1.4** and Figures **C.2.1** to **C.2.4** are based on different theories from those on which **Equations C.6** and **C.9** are based. The two methods yield different results when δ is not equal to zero.

The other comment on the application of **Annex C** given in the UK National Annex (published in December 2007) is no longer required as corrections have been made to Annex C through BS EN 1997-1:2004 Corrigendum no. 1. It will be removed from the UK National Annex when it is next updated.

Values of Partial, Correlation, and Model Factors for Ultimate Limit States to Be Used in EN 1997-1:2004 (Appendix 1)

National determined parameters (A.1)

Annex A contains the values of partial factors for geotechnical actions (γ_F) or the effects of geotechnical actions (γ_E), soil properties (γ_M) and resistances (γ_R), and also correlation factors (ξ) for pile foundations and advice on the use of model factors. The terminology generally follows that used in BS EN1990:2002. However, this is not always the case, as explained below, and where it is not, the National Annex has been drafted to clarify the relationship between the terminology used in BS EN 1997-1:2004 and BS EN1990:2002.

As noted in the comments on **Annex B**, above, BS EN1997-1:2004 is not always consistent in its treatment of situations where no partial factor is applied to a particular parameter and cases where a partial factor equal to 1.0 is applied. **Clause A.1.2** of the National Annex aims to explain how the specified partial factors should be applied. BS EN1997-1:2004 provides the opportunity for the National Annex to do so in NOTE 1 to clause **2.4.7.3.4.1(1)P**. Equations (**2.7a**) and (**2.7b**) are simplified versions of Equation (**2.7c**) which may be used in situations where $\gamma_R = 1$ and $\gamma_M = 1$ respectively. Similarly, for sliding, Equations **6.3a** and **6.4a** are simplified versions of the full expressions for sliding resistance for situations where $\gamma_{R;h} = 1$ and Equations **6.3b** and **6.4b** are simplified versions of the full expressions for sliding resistance for situations where $\gamma_M = 1$.

There is therefore a need for care in the application of some clauses which might, at first reading, appear to give a choice about the equations to be used for a particular verification, but on closer examination do not. Such potential ambiguity arises in BS EN 1997-1:2004, **6.5.3(8)P** and **6.5.3(11)P**. Clause **6.5.3(8)P** states that, "the design shear resistance, R_d, shall be calculated either by factoring the ground properties or the ground resistances". Equations (**6.3a**) and (**6.3b**), are provided for these two cases. Read in isolation, it might appear that the designer can choose which of the two equations to apply. However, the choice of which equation to use depends, in fact, on which Design Approaches is adopted. For Design Approach 1, used in UK, the simplified expressions given in Equations (**6.3a**) and (**6.4a**) should always be used because $\gamma_{R;h} = 1$. The use of Equations (**6.3b**) and (**6.4b**) would be potentially unsafe.

Partial factors for the equilibrium limit state (EQU) verification (A.2)

Partial factors on actions (γ_F) (A.2.1)

General considerations

Partial factors on actions are given in the National Annex to BS EN 1990:2002. BS EN1990:2002 provides different factors for buildings and for bridges, with recommended values given in **Annex A1** and **Annex A2** respectively. The UK National Annex for BS EN1990:2002 gives the values to be used in UK.

The terminology used for the partial factors on actions differs between BS EN1990:2002 and BS EN1997-1:2004. In clause **A.2.1** and table **A.NA.1**, the National Annex to BS EN1997-1:2004 seeks to address the potential for ambiguity by referencing the relevant tables from the UK National Annex to BS EN 1990:2002 as the source of the relevant partial factors. It also highlights that the terms $\gamma_{G;sup}$ and $\gamma_{G;inf}$ in BS EN 1990:2002 correspond with $\gamma_{G;dst}$ and $\gamma_{G;stb}$ in BS EN 1997-1:2004.

Clause **A.2.1** highlights that the specified partial factors can give an overall factor of safety on overturning lower than that for which confidence has been gained through past UK practice. Usually this concern will not arise because the bearing resistance of the ground will govern, but it will be advisable for designers to satisfy themselves that they are comfortable with the overall level of safety being achieved against overturning in special cases when the bearing resistance of the ground or the strength of the structure does not govern.

Water pressures and earth pressures

Clause **A.2.1** states that the partial factors specified in the National Annex to BS EN 1990:2002 might not be appropriate for self-weight of water, ground-water pressure and other actions dependent on the level of water. It also explains that the design value of such actions may be directly assessed in accordance with **2.4.6.1(2)P** and **2.4.6.1(6)P** of BS EN 1997-1:2004, or alternatively, a safety margin may be applied to the characteristic water level, see **2.4.6.1(8)** of BS EN 1997-1:2004.

There are several clauses in BS EN1997-1:2004 that relate to the treatment of water pressures. Amongst these: clause **2.4.4(1)P** states that water levels are treated as geometrical data; clause **2.4.5.3(1)P** states that, "Characteristic values of levels of …ground-water or free water shall be measured, nominal or estimated upper or lower levels"; clause **2.4.6.3(2)P** states that, "design values of geometrical data shall either be assessed directly or be derived from nominal values"; clause **2.4.6.1(2)P** states that, "The design value of an action … shall either be assessed directly or shall be derived from representative values"; **clause 2.4.6.1(6)P** states generally that for ultimate limit states the design values for ground water pressures, "shall represent the most unfavourable values that could occur during the design lifetime of the structure" and that for serviceability limit states the design values are, "the most unfavourable values which could occur in normal circumstances"; clause **2.4.6.1(8)** states that the "Design values of ground-water pressures may be derived either by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level"; and, clause **2.4.7.3.2(2)** states that, "In some design situations, the application of partial factors to actions coming from or through the soil (such as earth or water pressure) could lead to design values, which are unreasonable or even physically impossible. In these situations, the factors may be applied directly to the effects of actions derived from representative values of the actions."

In summary therefore, designers have five choices for how to determine the design value for actions arising from self-weight of water, ground water pressures, and other actions dependent upon the level of water. The design values may either be:

(i) based on directly determined values,
(ii) determined by applying a (dimensional) safety margin to characteristic water level,
(iii) determined by applying a partial factor to the characteristic water pressure,
(iv) a mixture of (ii) and (iii), as suggested by Bond and Harris[3], or
(v) following **2.4.7.3.2(2)**, apply the load factors to the effects of the water actions, *i.e.* to derived bending moments and forces in the supporting structure..

As highlighted by BS EN 1997-1:2004, **2.4.7.3.2(2)** there can be difficulties associated with calibrating partial factors to apply to water pressures, particularly hydrostatic pressures. This problem is compounded by the fact that increasing water pressure in the ground often has a double effect: it causes a reduction in strength, as well as an increase in disturbing force. Uncertainty in the magnitude of hydrostatic water pressures is primarily associated with the water level. Applying a partial factor to the characteristic water pressure can generate the following specific issues (although for the reasons discussed further below it can still be an entirely appropriate approach to take in design):

a. the design water pressure is only applied below the characteristic level, rather than the "most unfavourable" level (see **2.4.6.1(6)P**);
b. the value of design water pressure can imply a total depth of water that would overtop a retaining structure, making it "physically impossible"; and,
c. the resulting variation of design water pressure with depth might be considered to imply a "physically impossible" water density.

It is interesting to note that points b and c above can be equally valid criticisms of applying partial factors to the characteristic weight density of the ground.

Because of the resulting difficulties of calibration, the recommended values for partial factors for hydrostatic pressures given in BS EN1990:2002, **Annex A2** for bridges have not be retained in the UK National Annex. If partial factors are applied to characteristic water pressures, the appropriate values of such factors should be established on a project specific basis. Since they will be sensitive to the particular circumstances in which they are applied, no specific values are provided in the UK National Annex to BS EN1990:2002.

It is important to note that whilst the application of a partial factor to water pressures might have the potential to appear to give physically unrealistic results, it is not necessarily an inappropriate approach. In fact, in some cases, it can be rather important to factor water pressures to achieve a suitably safe overall design, as discussed below.

In cases where the effects of water pressure are small, as will usually be the case in bridge design (if they are present at all), the design will be insensitive to the approach used, and it follows therefore that it is not too important which approach is taken. Past practice has generally been to use 'directly determined' cautious values as in approach (i) above, without applying a partial factor, and this remains an available option for designs to Eurocodes.

However, in cases where the effects of water pressure are significant, for example where the depth of water might overtop a retaining structure, taking the maximum water level equal to the top of the structure in conjunction with a partial factor of unity can result in a low overall level of safety.

As discussed by Denton et al[8], the partial factors on actions (γ_F) given in BS EN1990:2002, **Annex A2** account for two sources of uncertainty. The first is the possible deviation in the value of action itself (denoted γ_f), and the second is uncertainty in modeling the effect of the action (denoted γ_{Sd}) (see BS EN 1990:2002, **6.3.2(2)**). Whilst in the case described above, it is likely to be reasonable to take $\gamma_f = 1.0$ since the water level is limited by the height of the retaining structure, it may be entirely sensible to retain a value of γ_{Sd} greater than unity. (The UK National Annex to BS EN1990:2002, **Annex A2** provides some general guidance on values of γ_{Sd}, see also PD 6694-1[13]).

For any particular design, it might be appropriate to check more than one of the methods (i) to (v) above. For example, with Design Approach 1, method (i) could be used in Combination 2, with method (iii) or (v) with Combination 1. For the latter check, the factor adopted might be reduced from γ_F to γ_{Sd}, as discussed above.

Design value of earth pressures
Finally, National Annex to BS EN 1997-1:2004 clause **A.2.1**, explains that the design value of earth pressures should be based on the design value of the actions giving rise to the earth pressure. This has important implications for bridge design. It means that earth pressures due to traffic surcharge should be factored using the relevant partial factors applicable to traffic load. It also explains that an additional model factor might be required in evaluating horizontal earth pressures for bridge design, see discussion on NA clause **A.6**.

Partial factors for soil parameters (γ_M) (**A.2.2**)

The partial factors for soil parameters for the Equilibrium limit state (EQU) are given in Table **A.NA.2**. The values in the National Annex are slightly lower than the recommended values in BS EN1997-1:2004. This was done to avoid EQU governing in cases in which it was not intended to do so (see also Denton et al[8], Schuppener et al [14]). The same factors are applicable in both building and bridge design.

The Note in Table **A.NA.2** highlights that if the reciprocal value of the specified partial factor produces a more onerous effect than the specified value, the reciprocal value should be used.

This can be particularly relevant in the case of bridge design. Dividing the coefficient of shearing resistance, tan φ', by a partial factor greater than unity reduces passive pressures and increases active pressures, and usually, this will lead to the critical case design case. However, there are special cases where active pressures are favourable, for example in the design of a buried concrete box where the worst case for sagging in the deck slab is typically considered with (favourable) active pressures applied to the walls of the box. Similarly, for integral bridge design, passive pressures on the abutments can be unfavourable.

The Note in Table **A.NA.2** does however highlight the single source principle, by reference to the NOTE below BS EN 1997-1:2004, **2.4.2(9)P**, whereby unfavourable and favourable permanent actions arising from a single source may have the single partial factor applied to them. This reference was included to highlight that if, for example, earth pressures applied to a structure from a single source are partially favourable and partially unfavourable, it may be appropriate to use a single value of γ_M (see PD6694-1[13]).

Partial factors for structural (STR) and geotechnical (GEO) limit states verification (A.3)

Partial factors on actions (γ_F) or the effects of actions (γ_E) (**A.3.1**)

As explained in respect of **A.2.1** above, partial factors on actions are given in the National Annex to EN 1990:2002. EN1990:2002 provides different factors for buildings and for bridges, with recommended values given in **Annex A1** and **Annex A2** respectively. However, as discussed above, the terminology used for the sets of partial factors on actions differs between BS EN1990:2002 and BS EN1997-1:2004.

For verification of Structural (STR) and Geotechnical (GEO) limit states, Design Approach 1 requires calculations to be undertaken with two different sets of partial factors. Table **A.NA.2** makes reference to the relevant tables in the UK National Annex to BS EN 1990:2002 as the source of the partial factors on actions. The table show the relationship between BS EN 1997-1:2004 Set A1 and BS EN 1990:2002 Set B, and between BS EN 1997-1:2004 Set A2 and BS EN 1990:2002 Set C, as illustrated in Figure 1(a) and (b).

National Annex clause **A.3.1** repeats the clauses from **A.2.1** relating to the effects of self-weight of water, ground water and other actions dependent on the level of water, and relating to design values of earth pressures. The background to these is explained above.

Partial factors for soil parameters (γ_M) (**A.3.2**)

The partial factors for soil parameters for the Structural (STR) and Geotechnical (GEO) limit states are given in Table **A.NA.4**. The recommended values in EN1997-1:2004 value have been retained, with the same factors applicable to both building and bridge design. In addition, a partial factor for the critical state angle of shear resistance is provided, although it is noted that it may be more appropriate to assess the design value of tan φ'_{cv} directly rather than to apply a partial factor.

The use of φ'_{cv} is specifically required by BS EN 1997-1:2004 to limit interface friction between structures and soil (6.5.3(10), 9.5.1(6)). Guidance on its value can be found in PD6694-1. It is often possible to be more confident about the value of φ'_{cv} than of any higher values that might be operative in more compact soil. Furthermore, φ' for compact soil tends to fall to φ'_{cv} as strains become large, which would usually happen if an ultimate limit state developed. It is therefore arguable that the *design* value of φ'_{cv} for ULS calculations could be assessed directly, without the need for a further reduction factor. If this is done, however, it is important to bear in mind that the partial factors applied to loads and materials in reality have to provide a sufficient margin to cover other minor eventualities such as imperfections in geometry or other features of construction. It is therefore important that a sufficient margin remains for these.

The note in Table **A.NA.4** is identical to that in Table **A.NA.2**, the background to which is explained above.

Partial resistance factors (γ_R) (**A.3.3**)

Clause **A.3.3** of the National Annex contains partial resistance factors and also correlation factors for pile foundations.

Partial resistance factors for spread foundations, retaining structures, and slopes and overall stability are given in National Annex **Tables A.5**, **A.13** and **A.14**, respectively. The recommended values from BS EN1997-1:2004 have been retained, although the tables have been edited to contain only those values relevant to Design Approach 1. The relevant partial factors are called Set R1, for which in all cases, the partial factor is 1.0.

The background to the partial resistance factors and correlations factors for piles is given by Bond and Simpson[11],[12].

A.4 Partial factors for the uplift limit state (UPL) verification

Partial factors on actions (γ_F) (**A.4.1**)

The uplift limit state concerns a loss of equilibrium of the structure or ground due to uplift by water pressure or other vertical actions (see BS EN 1997-1:2004, **2.4.7.1(1)P**). Although it will rarely be critical to structural design, designers should satisfy themselves that members have adequate structural resistance to sustain action effects arising at UPL. In bridge design, the uplift limit state is therefore generally very unlikely to be of concern.

Table **A.NA.15** gives the partial factors on actions for the uplift limit state. The recommended values from BS EN1997-1:2004 have been retained, except that the factor on unfavourable permanent actions has been increased to 1.1 to bring it in line with UK National Annex to BS EN 1990:2002, **Table NA.A1.2(A)**, and, for completeness, a partial factor of 0.0 is also included for variable favourable actions.

The partial factors given in **Table A.NA.15** therefore follow those in UK National Annex to BS EN 1990:2002, **Table NA.A1.2(A)**, which contains the EQU partial factors for buildings. It was not considered necessary to include an alternative bridge specific table in the National Annex because the uplift limit state is unlikely to be relevant in bridge design, and also because the specified values are more onerous than the values in UK National Annex to BS EN 1990:2002, **Table NA.A2.4(A)** which might otherwise have been used as the basis for a bridge specific table.

For similar reasons to those set out in relation to **A.2.1**, the Note in **Table A.NA.15** explains that the partial factor for permanent unfavourable actions does not account for uncertainty in the level of ground water or free water. For a fully submerged structure, the buoyant weight is independent of the depth to which it is submerged. However, for partially submerged structures the level of ground water or free water is significant.

Partial factors on soil parameters (γ_M) and resistances (γ_R) (**A.4.2**)

Table A.NA.16 gives the partial factors for soil parameters and resistances for the uplift limit state. The recommended values have been retained. Note 1 has been included to identify the need to use the reciprocal of the specified value of the partial factor on soil parameters where this has a more onerous effect.

Partial factors for actions for the hydraulic heave limit state (HYD) verification (A.5)

Table A.NA.17 gives the partial factors for actions for the hydraulic heave (HYD) limit state. The recommended values are retained except that, as with **Table A.NA.15** for completeness, a partial factor of 0.0 is also included for variable favourable actions. (Note that the partial factor on unfavourable permanent actions is given incorrectly as 1.335 rather than 1.35 in the December 2007 version. This typographical error will be corrected in a future corrigendum.)

For similar reasons as the uplift limit state, it was not considered necessary to include an alternative bridge-specific table in the National Annex.

BS EN 1997-1:2004, **2.4.7.5(1)P** gives two criteria that may be used for verifying the hydraulic heave limit state. These are expressed as equations (**2.9a**) and (**2.9b**). Equation (**2.9a**) requires that the destabilising total pore water pressure is less than or equal to the stabilising total vertical stress whilst equation (**2.9b**) requires that, for a column of soil, the seepage force is less than or equal to the column's submerged weight.

Although these two conditions are effectively equivalent, they can yield different results depending upon the way in which partial factors are applied. The difficulty arises because although Table A.NA.17 gives values for the factors, BS EN 1997-1 does not state where they are to be applied in calculations, which has led to confusion (Orr[15]). A pore water pressure component appears on both sides of equations (**2.9a**), whereas it does not appear on either side of equation (**2.9b**). Thus, if the partial factor on stabilising permanent actions is applied to all of the components of the stabilising total vertical stress and the partial factor on destabilising actions is applied to all of the components of the destabilising total pore water pressure it has the effect of applying different partial factors to the pore water pressure components on either side of equation (**2.9a**). Since the pore water pressure component of the destabilising total pore water pressure and the stabilising total vertical stress arises from the same source (in fact, being the same pressure), such a differentiation of partial factors is unnecessary according to the single source principle, see NOTE below BS EN 1997-1:2004, **2.4.2(9)P**.

The Note in Table **A.NA.17** explains that the single source principle may be applied to the hydrostatic component of the total pore water pressure and the total vertical stress, thereby overcoming the potential for equations (**2.9a**) and (**2.9b**) to yield different results.

Model factors (A.6)

Clause **A.6** of the National Annex includes guidance on the application of model factors. BS EN1997-1:2004 makes several references to the use of model factors. Clause **A.6.2** explains that for buildings designed using conventional calculation methods, it may be assumed that the necessary model factors are incorporated in the partial factors given in Annex A of the National Annex, with the exception of a few specific cases that are detailed in **A.6.5** and **A.6.6**.

However, bridges have traditionally been designed using higher factors of safety for some geotechnical aspects than buildings. Clause **A.6.3** of the National Annex therefore highlights that additional model factors may be required for some aspects of bridge design to maintain current safety levels. Guidance on this is provided in PD6694-1[13] (see also Denton et al[16]).

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PD 6694-1: Recommendations for the Design of Structures Subject to Traffic Loading to EN 1997-1

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Abstract
This paper gives the background to the development of the provisions of PD 6694-1. It gives guidance on the application of PD 6694-1 where it is considered that further explanation may be helpful and identifies recommendations in PD 6694-1 which involve design principles or procedures significantly different from those used in past practice.
The paper covers the clauses in the PD 6694-1 relating to actions, spread foundations, buried structures and earth pressure on gravity retaining structures and bridge abutments. Traffic surcharge and integral bridges are covered in detail in companion papers, for which references are provided.

Notation
The same notation is used as in the Eurocodes and PD 6694-1. Other symbols are defined within the clause in which they occur.

The Clause numbers used in the headings of this paper are the Clause numbers in PD 6694-1 to which the text refers.

Introduction
The recommendations given in PD 6694-1 (hereafter referred to as “the PD”) apply to structures that are subject to traffic surcharge and other traffic loading. The recommendations therefore specifically relate to the rules and partial factors given for "bridges" as opposed to "buildings" in the Eurocodes. Many of the principles described can however be applied to earth retaining structures that are not subject to traffic loading.

BS EN 1997-1:2004 does not specifically cover aspects of design of some types of highway and rail structures such as integral bridges and buried structures. Complementary design recommendations and guidance is therefore included in the PD.

For highway structures, PD 6694-1 replaces BD 30/87[2] (Earth Retaining Structures), BD 31/01[3] Buried Structures, BA 42/96[1] Integral Bridges and BD 74/00[5] Foundations. The design of reinforced earth structures is neither covered in BS EN 1997-1:2004 nor in the PD.

Basis of Design (4)

Dispersion of vertical load through fill (4.4)

The justification for the 30° method of dispersing vertical loads is given later in this paper in relation to buried structure (**10.2.7**). The use of the 30° dispersion method may however be unsafe when the vertical pressures arising from it are favourable. For example, where sliding resistance is dependent on the load on the base slab, it may be unsafe to assume that part of the weight of the surcharge traffic behind the abutment is supported on the base slab because other dispersion modes including vertical soil arching can occur which may result in the vertical load being supported on the ground behind the base slab while the horizontal surcharge effect is still applied to the wall.

When analysing the foundations for bearing pressure the vertical pressure on an abutment base slab due to the traffic surcharge may be favourable or unfavourable. In some cases the additional pressure may increase the toe pressures, but in other cases it may apply a large enough restoring moment to reduce the toe pressure. If the effect of the vertical pressure from traffic surcharge is favourable in respect of bearing pressure, it may be prudent to ignore it.

Model Factors on horizontal earth pressure at ULS (4.7)

Following the publication of BS EN 1997-1:2004, concern was expressed that the ULS partial factors were significantly lower than those used in pre-Eurocode standards for bridge design. In particular it was seen that the effective ULS partial factor γ_{fL}. γ_{f3} specified for horizontal earth pressure in BD 37/01[4] equalled 1.5x1.1 = 1.65 compared with a γ_F of 1.35 for the critical STR/GEO limit state, Design Approach 1, Combination 1 partial factors in the Eurocode (*i.e.* Set A1 in BS EN 1997-1:2004 and Set B in BS EN 1990:2002). This would mean that structures designed to BS EN 1997-1:2004 could be less robust than those designed in the past.

To address this concern, the PD states that where it is required to maintain the same levels of safety as were applied in the past, a model factor $\gamma_{Sd;K}$ may be applied to the horizontal earth pressure (effectively to K_a or K_0). The recommended value of the model factor was based on the ratio of the pre-Eurocode factors to the STR/GEO Combination 1 factor, namely 1.65/1.35 = 1.22 (rounded down to 1.2), to give similar design values for earth pressures. Its effect was examined for other ultimate limit states verifications.

For sliding and overturning, BD 30/87, **5.2.4.2**[2] references CP 2[8] in which it says, in relation to sliding: "…a factor of safety of approximately 2 should be applied…" and "…the angle of friction below the base is equal to ϕ, the angle of friction of the soil beneath the foundation".

On this basis, the required heel length B_{heel} for an abutment of height Z is given by:

$$B_{heel} = 2K_{a;k}\{\gamma Z^2/(2\tan\phi')\}$$

For a Eurocode design using the model factor $\gamma_{S;dK}$:

$$B_{heel} = \gamma_{Sd;K}K_{a;d}\{\gamma Z^2/(2\tan\phi'_{cv})\}$$

From this is can be shown that, using the model factor and the relevant values of the partial factors, the Eurocode value of B_{heel} will not be less than the pre-Eurocode value if $\tan\phi'_{\text{cv}}$ is not greater than about 0.9tan ϕ'. In practice $\tan\phi'_{\text{cv}}$ is almost invariably less than $0.9\tan\phi'$.

For sliding resistance of an undrained foundation CP 2[8] uses a similar method to the Eurocode. For the CP 2[8] method with a factor of safety 2 on sliding: $2H = Bc_u$ where B is the base length, Z is the height of the wall and the horizontal action $H = K_{a;k}\gamma Z^2/2$. Thus,

$$B = 2K_{a;k}\{\gamma Z^2/(2c_u)\}$$

In the Eurocode, for a retaining wall subject to permanent actions and the model factor, $\gamma_G\gamma_{Sd;k}H = Bc_u/\gamma_M$ where in Design Approach 1, Combination 2 $\gamma_G = 1$ and $\gamma_M = 1.4$.

$$B = 1.4\ \gamma_{Sd;K}\mathrm{K}_{a;d}\{\gamma Z^2/2c_u\}$$

From this it can be shown that based on $\phi'_k = 33°$ for the backfill and the relevant values of the partial factors, the Eurocode base length will be approximately 5% longer than the CP 2[8] base length if the model factor is included, and approximately 13% shorter if the model factor is not applied.

The above comparisons apply to retaining walls subject to permanent earth pressure only. When surcharge, braking and acceleration are applied, the pre-Eurocode base lengths will theoretically be relatively longer. In practice though, the Eurocode surcharge action is so much larger than the pre-Eurocode surcharge action that it is unlikely that base slabs subject to the Eurocode surcharge will be shorter than base slabs designed in the past.

Bearing resistance is frequently governed by settlement requirements at SLS for which the ULS model factor is irrelevant. For ultimate bearing resistance it is less easy to make a direct comparison between Eurocode and pre-Eurocode designs because of the number of different acceptable pre-Eurocode design methods available. Specimen comparative calculations have however shown that if the model factor is applied to the horizontal earth pressure, the Eurocode designs for bearing resistance will usually be comparable with pre-Eurocode designs.

In relation to overturning, CP 2[8] says "...in gravity walls the resultant thrust should not fall outside the middle third of the base, and for other types of wall a factor of safety of at least 2 against overturning is required". Overturning is not usually an issue with conventional gravity walls and abutments because the bearing resistance under the toe will normally become critical before the structure overturns and the length of heel required to provided sliding resistance is usually sufficient to give an adequate restoring moment. Overturning could however become an issue with a mass gravity wall seated on rock or a concrete slab and propped or keyed into the slab to prevent sliding as shown in Figure 1.

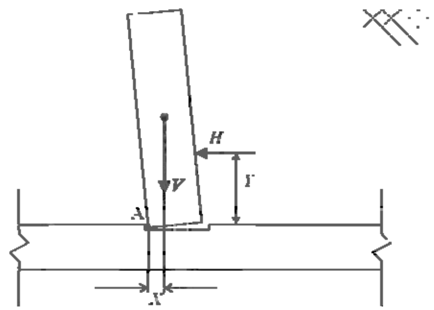


Figure 1

For the above structure, considering overturning about A at ULS, the Eurocode effectively requires that the maximum design overturning moment should not be greater than the minimum design restoring moment:

$$Y\,H\,\gamma_{G;soil;sup}\gamma_{Sd;k}\gamma_{MK} \le X\,V\,\gamma_{G;conc;inf}$$

where H and V are characteristic actions and $\gamma_{MK} = K_{a;d}/K_{a;k} \approx 1.11$ at EQU and 1.25 at STR/GEO combination 2 if ϕ' is about 33°.

The overall factor of safety is XV/YH which equals $(\gamma_{G;soil;sup}\gamma_{Sd;K}\gamma_{MK})/(\gamma_{G;conc;inf})$.

This equals (1.05 x 1.2 x 1.11/0.95) = 1.47 at EQU and (1.35 x 1.2 x 1.0/0.95) = 1.70 at STR/GEO Combination 1.

These values reduce to 1.23 and 1.42 respectively if the model factor $\gamma_{Sd;K}$ is not applied. However, it can be shown that if this structure was designed to comply with the "middle third" rule at SLS then the factor of safety would automatically be ≥ 3.0.

From the above comparisons it can be seen that the 1.2 ULS model factor compensates for the difference between the Eurocode and pre-Eurocode values of ULS partial factors in relation to earth pressure, sliding resistance and ultimate bearing resistance, and it is irrelevant in regards to settlement and overturning except in the unusual situation where a structure such as that shown in Figure 1 is not designed to comply with the middle-third requirement at SLS.

The Eurocode surcharge loading for highway structures is substantially more onerous than the HA and HB surcharge used in the past, and as this will result in stronger rather than weaker structures, the 1.2 model factor is not required to be applied to the effects of traffic surcharge loading.

The PD does not offer an opinion as to whether the pre-Eurocode standards were unduly conservative. The option to use the model factor is for designers and clients who wish to maintain past levels of safety in their earth retaining structures.

Pile Foundations (6)

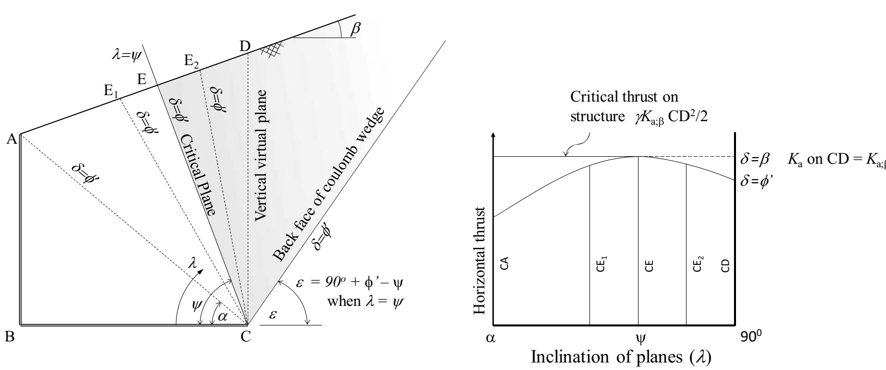
This clause is based on the recommendations in BD 74/00 Annex B[5].

Gravity Retaining Structures and Bridge Abutments (7)

Earth pressures on retaining walls and abutments with inclined backfill (7.2.3 and 7.2.4)

Active pressure on walls with long heels ($\alpha<\psi$) (7.2.3)

Figure 2(a) shows a structure retaining fill with a surface inclined at an angle β. Figure 2(b) shows a plot of the horizontal thrust on plane CE as its inclination, λ, increases from α to 90°. As can be seen the thrust reaches a peak when $\lambda = \psi$. In Appendix 1 to this paper it is shown that this peak thrust on CE (and therefore the horizontal thrust on the whole structure) is the same as the thrust on the vertical virtual face CD when δ on that face equals β. This means that provided α is less than ψ, the total horizontal thrust on the structure can be found by calculating the horizontal thrust on CD taking $\delta = \beta$. Appendix 1 also provides a proof of the equation for ψ given in the PD (which is the same as the corrected expression given in Clayton and Milititsky[9]). Values of ψ based on this equation are shown in Figure 3. ψ is not however used in the pressure calculation except to determine the minimum length of BC, (AB$\cot\psi$), for which the $\delta = \beta$ method is valid.



(a) Retaining wall with inclined fill

(b) Variation of horizontal thrust on virtual planes of differing inclinations when δ on these planes equals ϕ′

Figure 2. Horizontal thrust on inclined backfill planes

Example 1 (Long heel)

A 6m high wall with a 4m heel supports backfill inclined at 20°. Find the active characteristic horizontal thrust applied to the structure if $\gamma = 18$ kN/m^3 and $\phi' = 35°$:

When $\delta = \beta = 20^o$, K_a from PD 6694-1, **Table 4** equals 0.30
Height of virtual face CD = 6 + 4tan20^o = 7.46m
Horizontal thrust = $\gamma K_a CD^2/2$ = 18x0.3x7.46²/2 = 150.3kN/m width.

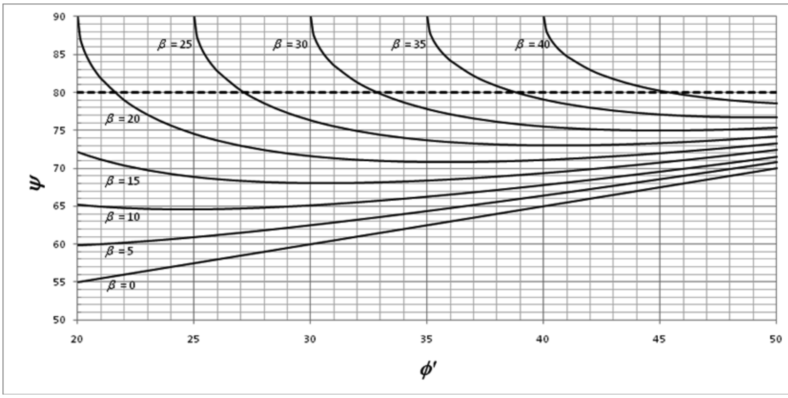


Figure 3. Values of ψ

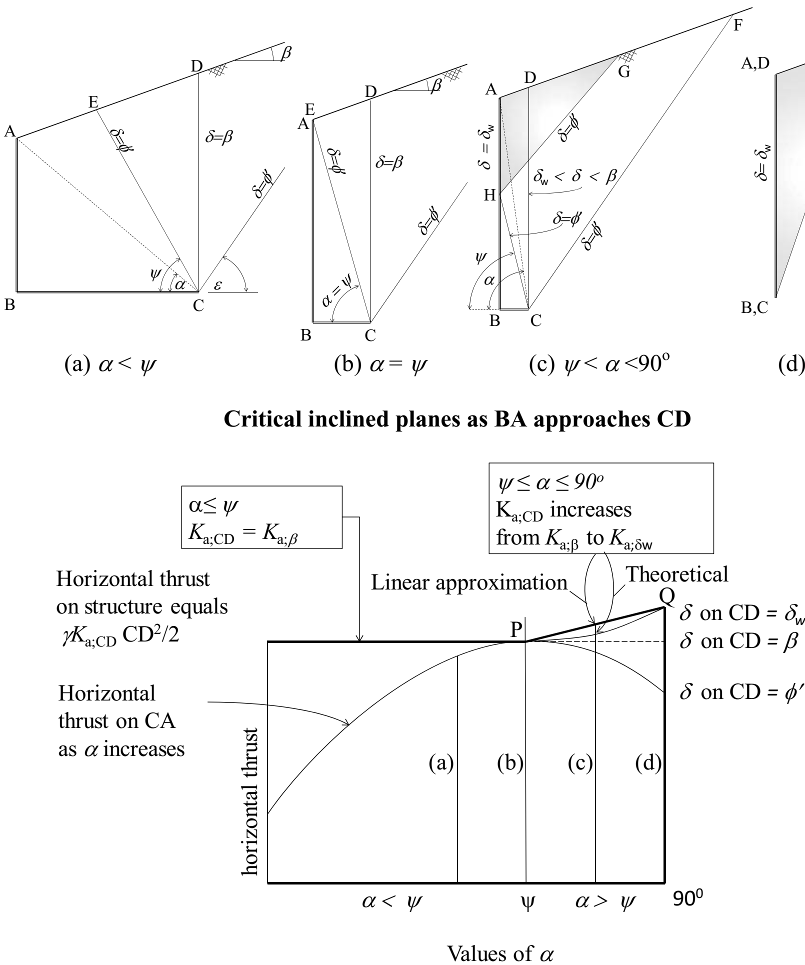
Active pressure on walls with short heels $\alpha > \psi$ (7.2.4)

When $\alpha > \psi$, (that is when BC is less than ABcotψ as in Figure 4(c)) the critical inclined virtual face CE is interrupted by the back face of the wall at H as shown in Figure 4(c) and the results described in the previous paragraph will no longer apply because AH in Figure 4(c) is a soil-to-wall surface and δ over that length will be δ_w rather than ϕ'.

When $\alpha = \psi$, as in Figure 4(b) δ on CD = β. When $\alpha = 90^o$, as in Figure 4(d), δ on CD equals δ_w because CD is then coincident with BA and δ_w is applied over the full height of the wall. When α lies between ψ and 90^o, the effective value of δ on CD therefore lies between β and δ_w, and the effective value of K_a on CD (that is $K_{a;CD}$) will lie between $K_{a;\beta}$ and $K_{a;\delta w}$ as described in the PD.

To determine the value of K_a on CD for an intermediate position with $\psi < \alpha < 90$, the shaded triangle AHG in Figure 4(c) was considered as a Coulomb wedge with $\delta = \delta_w$ on AH, and HCFG was considered as a four-sided wedge with $\delta = \phi'$ on all soil-to-soil faces. Using this model for a number of values of α it was found that although theoretically as α increased from ψ to 90^o the plot of $K_{a;CD}$ followed a curve between $K_{a;\beta}$ and $K_{a;\delta w}$, in practice it was simpler and marginally conservative to assume that $K_{a;CD}$ increased linearly with α from $K_{a;\beta}$ to $K_{a;\delta w}$ in this range.

Figure 4(e) plots the value of the thrust on plane CA and on the whole structure as the wall BA moves towards CD and α increases from less than ψ to 90^o. The arc PQ shows the theoretical increase in effective $K_{a;CD}$ as α increases from ψ to 90^o and the chord PQ shows the linear variation in $K_{a;CD}$ assumed in the PD.



(e) Horizontal thrusts as BA moves towards CD

Figure 4. Abutments supporting inclined backfill

In the above paragraphs it is assumed that δ_w is less than β. If δ_w is greater than β, then $K_{a;\beta}$ is greater than $K_{a;\delta w}$ and $K_{a;CD}$ theoretically *reduces* from $K_{a;\beta}$ to $K_{a;\delta w}$ as α increases from ψ to 90°. As this effect is small and only significant when α is very close to 90°, the PD recommends that when δ_w is greater than β, K_a on CD should be taken as $K_{a;\beta}$ for all values of α.

Example 2 (Short heel)

A 6m high wall with a 1.5m heel supports backfill inclined at 20°. Find the characteristic active pressure applied to the structure if γ = 18 kN/m^3 and ϕ'= 35° and δ_w is taken conservatively as 10°:

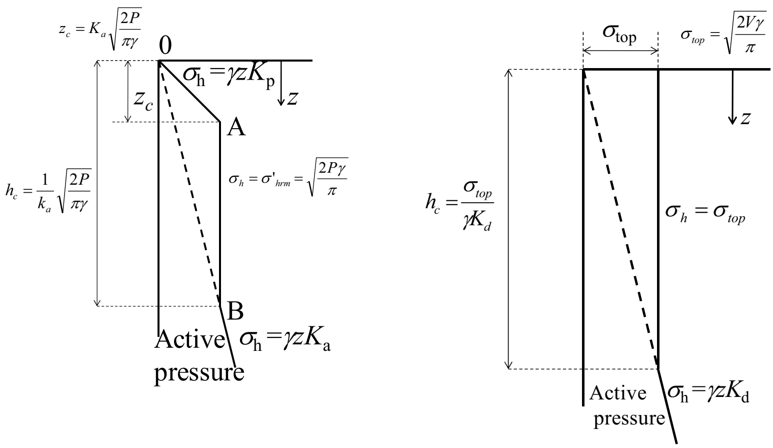
$\alpha = \tan^{-1}(6/1.5) = 76°$
ψ = from Figure 3 (above) (for β = 20° and ϕ'= 35°) = 70.8°. $\alpha > \psi$
When $\delta = \beta = 20°$, $K_{a;\beta}$ from PD 6694-1, **Table 4** equals 0.302
When $\delta = \delta_w = 10°$ and $\beta = 20°$, $K_{a;\delta w}$ by wedge analysis or other means = 0.322
Increase in $K_{a;CD} = (K_{a;\delta w} - K_{a;\beta})\{(\alpha-\psi)/(90-\psi)\} = (0.322-0.302)\{(76-70.8)/(90-70.8)\} = 0.005$
$K_{a;CD} = 0.302+0.005 = 0.307$
Height CD = 6 + 1.5tan20 =6.55m
Horizontal thrust = $\gamma K_a CD^2/2 = \gamma K_a 6.55^2/2 = 18 \times 0.307 \times 6.55^2/2$ = 118.5kN/m width

Walls with both α and ψ greater than 80°

It can be seen from the equation for ψ that when β approaches ϕ', ψ approaches 90°. It has been found that in the unusual situation when both α and ψ are greater than 80° (i.e. steep backfill with a very small heel), the value of $K_{a;CD}$ starts to increase towards $K_{a;\delta\omega}$ when α is approximately 80° (*i.e.* less than ψ). The PD covers this situation by artificially limiting the maximum value of ψ to 80° in **7.2.4**. This limitation only affects the heel length at which the short heel effect becomes critical. It does not affect the value $K_{a;\beta}$.

Compaction pressures (7.3.4)

The pressure distribution shown by Clayton and Milititsky[9], Ingold[10] and others is reproduced in Figure 5(a). In practice the position of (A) usually occurs only a short way below the surface and it is simplest and only slightly conservative to consider the combined active and compaction pressure having the value σ_{top} given in the PD and being constant from ground level to the level at which active pressure equals σ_{top} as shown in Figure 5(b).



(a) Clayton[9] & Ingold[11] **(b) PD 6694-1**

Figure 5. Compaction pressures

Movement Required to Generate Passive Pressure (7.5)

BS EN 1997-1:2004, Annex C effectively defines the relationship between mobilised passive pressure and wall movement by three points and one gradient (see BS EN 1997-1:2004, Annex C, **Figure C.4**). The three points are K_o and zero movement, $0.5K_p$ and a movement of v_2/h and K_p and a movement of v/h. By the nature of passive pressure the gradient of the pressure/movement curve at full passive pressure is horizontal. This is illustrated in Figure 6.

In devising a formula to replicate the curve illustrated in **Figure C.4** in Annex C it was considered adequate to assume that the relationship was linear between K_o and $K_p/2$ and the curve between $K_p/2$ and K_p was a cubic curve passing through the $K_p/2$ and K_p points and having a horizontal gradient at K_p. The equation given in the PD achieves this. Inevitably in some cases the gradient of the curve at $K_p/2$ is not identical to the gradient of the K_o-$K_p/2$ line. The errors resulting from this are considered to be small compared to the tolerance on v_p/h and v_2/h given in BS EN 1997-1:2004, **Annex C**.

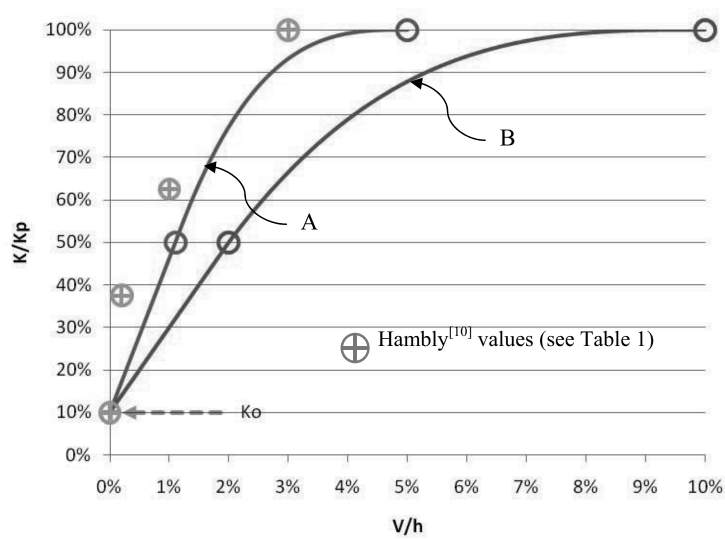


Figure 6. Plots of *K*/*K*$_p$ against *v*/*h* using the empirical equation in the PD clause 7.5

Plots A and B in Figure 6 are based on the maximum and minimum values of v/h for $K=K_p$ and half K_p given in section (a) of BS EN 1997-1:2004, **Table C.2.** It is clear from the range of figures quoted In **Table C.2** and Table1 below, that the relationship between movement and earth pressure can be very variable. The values of the wall rotations required to mobilise full passive and half passive pressure given in BS EN 1997-1:2004, **Annex C** are very large compared to those given by Hambly [10]. For example, to develop a "conventional" half K_p behind a 10m high wall the deflection at the top would be between 110mm and 200mm according to **Annex C** compared with about 50mm according to Hambly[10].

	Rotations required to mobilise Half K_p	Rotations required to mobilise Full K_p
Hambly[10]	0.5% (approx)	3% (approx)
Annex C to BS EN1997-1:2004	1.1% - 2%	5% - 10%

Table 1 Comparison of rotation required to mobilise passive pressure

Traffic surcharge (7.6)
The background to the surcharge model is described by Shave et al [13].

Integral Bridges (9)
See accompanying paper Denton et al [14].

Buried Concrete Structures (10)

This section (Clause 10) is based on BD 31/01[3] updated in line with the Eurocode requirements.

Superimposed permanent load (10.2.2)

The model factors $\gamma_{Sd;ec}$ to be applied for superimposed permanent load are taken from Figure 3.1 in BD 31/01[3].

Dispersal of vertical loads (10.2.7)

The dispersal of vertical loads through fill is given in the PD as 30° compared to 26.6° (2:1) in BD 31/01[3]. The 30° value is taken from BS EN 1991-2:2003, **4.9.1**, **NOTE 2**.

The Boussinesq equation given in clause 3.2.1 (iii) of BD 31/01[3] has been omitted from the PD as it underestimates the pressure when there is a rigid plane, such as the roof of a buried structure, located a short distance below ground level. The method in Table 2.1 of Poulos and Davis[12] gives pressures below a point load which are marginally lower than those found using the 30° dispersion method, as can be seen from Figure 7.

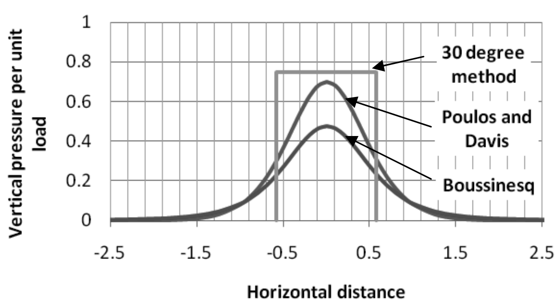


Figure 7. Comparison of the "30°" pressures with pressures based on the Poulos & Davis[12] and Boussinesq equations for 1 m fill depth.

Longitudinal road traffic actions (10.2.8)

Traffic surcharge (10.2.8.1)

The treatment of traffic surcharge for buried structures using the simplified model given in PD 6694-1, **7.6** is explained in **Table 5, Note B** of the PD. Its background is given by Shave et al[13].

Braking and acceleration (10.2.8.2)

The reduction of the braking and acceleration actions with increasing depth of earth cover given in PD 6694-1, **10.2.8.2** has been retained from BD 31/01[3] in the absence of data to justify a revaluation.

Longitudinal joints (10.5)

The 0.15H_c limitation of deflection on segmental units is taken from BD 31/01[3].

Soil Structure Interaction Analysis of Integral Bridges (Annex A to PD 6694-1)

See accompanying paper Denton et al[14].

Tables of Earth Pressures for Buried Structure (Annex B)

Maximum pressure (B.2 and tables B.1 and B.2)

The values of the earth pressure coefficients given in the tables in Annex A are based on the assumption that the maximum characteristic at rest earth pressure coefficient taking temperature and strain ratcheting into account, is 0.6, as in BD 31/01[3]. This may be considered as backfill with a characteristic ϕ' of 30° subject to an enhancement factor, F_{enh}, of 1.2 to allow for temperature, strain ratcheting and other unfavourable effects. In addition, at ULS, the pressure coefficients are subject to the model factor $\gamma_{Sd;K}$ (also equal to 1.2) described in **4.7**. Using the values of γ_M given in the UK National Annex to BS EN 1997-1:2004 the values of the earth pressure coefficient K_{max} given in **Tables B.1 and B.2** in Annex B are derived as follows:

	Characteristic	EQU	STR/GEO Comb 1	STR/GEO Comb 2
ϕ'_d (inc γ_M)	30°	27.7°	30°	24.8°
$K_0 = (1-\sin\phi'_d)$	0.5	0.53	0.5	0.58
K_0 x F_{enh}	0.6	0.64	0.60	0.70
$K_{max} = K_0 F_{enh} \gamma_{sdk}$	0.6	0.77	0.72	0.84

As traffic surcharge is not considered to be affected by temperature and strain ratcheting, and as $\gamma_{Sd;k}$ is not applied to traffic surcharge (see **7.6**) the design value of the at rest pressure coefficient for traffic surcharge given in PD 6694-1, Annex B **Tables B.1 and B.2** is simply the value of K_o given in the table above.

Minimum pressure (B.3 and Table B.3)

For members which are critical with minimum horizontal earth pressure, **Table B3** is relevant. The minimum characteristic earth pressure coefficient of 0.2 is as in BD 31/01[3]. This is the characteristic value of K_a for $\phi'_k = 30°$ multiplied by a reduction factor, $F_{red} = 0.6$. The values of $K_{min;d}$ given in the tables are based on the equation:

$$K_{min;d} = F_{red}(1-\sin\phi'_d)/(1+\sin\phi'_d)$$

where $\phi'_d = \tan^{-1}\{(\tan 30°)/\gamma_M{*}\}$ and $\gamma_M{*} = 1/\gamma_M$. This gives approximately the same result as taking $F_{red} = 1$, ϕ'_k as 38° with $K_{a;k}$ based on $\delta/\phi' = 0.66$ from BS EN 1997-1:2004, **Figure C.1.1** and $\gamma_M = \gamma_M{*}$ as before.

Figure C.1.1 gives values of K_a as low as 0.13 for $\phi' = 45°$ and $\delta = \phi'$ and it would therefore be prudent to ignore active pressure altogether if it was considered that backfills with high values of ϕ' were relevant.

Active pressure (B.4 and Tables B.4, B.5 and B.6)

When braking or acceleration actions are applied, active pressure based on a characteristic ϕ' of 30° is applied on the active face and K_{max} is applied to the passive face as in **Tables B.1** and **B.2**, except that as the single source principle in not applied at EQU and the K_{max} actions are favourable in resisting longitudinal actions, the characteristic value of K_{max} (= 0.6) is applied at EQU in **Tables B.4** and **B.5**.

Where it is necessary to increase the pressure on the passive face above the K_{max} pressure to resist the longitudinal traffic actions it should be noted that movements related to specific values of K given in **Table C.2** of Appendix C of BS EN 1997-1:2004 are substantially greater than those given by Hambly[10] (see Table 1 above).

References

[1] BA 42/96 Amendment No. 1(2003) *The design of Integral Bridges*, The Stationary Office, London
[2] BD 30/87 *Backfilled retaining walls and bridge abutments*, The Stationary Office, London
[3] BD 31/01 *The design of buried concrete box and portal frame structures*, The Stationary Office, London
[4] BD 37/01 *Loads for highways bridges*, The Stationary Office, London
[5] BD 74/00 *Foundations*, The Stationary Office, London
[6] BS EN 1991-2:2003 (incorporating Corrigenda December 2004 and February 2010), *Eurocode 1, Part 2, Traffic loads on bridges*, BSi, London, UK
[7] BS EN 1997-1:2004 (incorporating corrigendum February 2009) *Eurocode 7: Geotechnical design – Part 1: General rules*, BSi, London, UK
[8] CP 2 (1951) *Earth retaining structures*, BSi, London, UK
[9] Clayton, C. R. I and Milititsky, J. (1986) *Earth pressure and earth retaining structures* Surrey university Press, London
[10] Hambly, E.C. (1991) *Bridge Deck behaviour*, 2nd edition, London: E& FN Spon
[11] Ingold, T.S. (1979) *The effects of compaction on retaining walls*. Géothechnique 29(3), 265-283
[12] Poulous, H.G and Davis, E. H. (1974) *Elastic solutions for soil and rock mechanics* John Wiley & Sons, Inc, London
[13] Shave, J, Christie, T. J. C, Denton, S. and Kidd, A. (2010) *Development of traffic surcharge models for highway structures*, in Proceedings of Bridge Design to Eurocodes – UK Implementation, Ed. by S. Denton, Nov 2010, ICE, London.
[14] Denton, S, Riches, O, Christie, T. J. C. and Kidd, A. (2010) *Developments in integral bridge design*, in Proceedings of Bridge Design to Eurocodes – UK Implementation, Ed. by S. Denton, Nov 2010, ICE, London.

Appendix 1

Derivation of Earth Pressures on Walls With Long Heels and Sloping Backfill

This appendix provides a derivation of the critical wedge angle ψ, the friction angle δ to be used on the vertical virtual face, and the resulting earth pressure coefficient K_a, for a retaining structure such as that illustrated in Figure A1(a) where the backfill is sloping at an angle β, and the heel of the wall is long enough that $\psi > \alpha$.

Figure A1(b) shows the critical wedge ΔECF and the forces acting on it at the boundaries, R_1 and R_2, and the self-weight W.

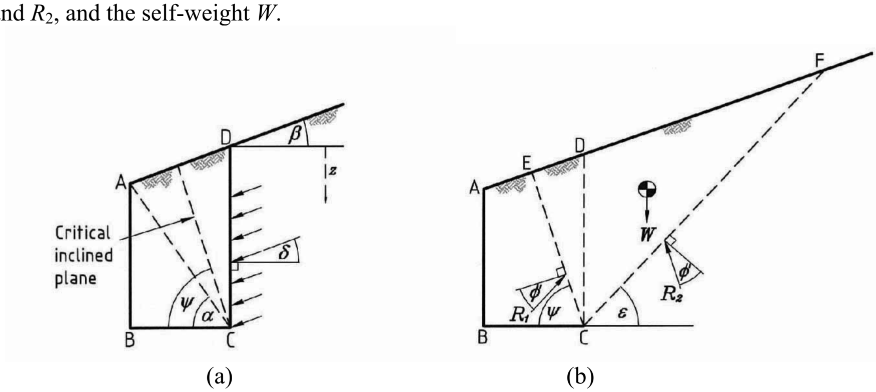


Figure A1. Earth pressures with sloping backfill

Figure A2 comprises the Mohr's circle for the critical wedge ΔECF as it reaches a critical state simultaneously along CE and CF (represented by points e and f). Point d represents the stress at the vertical plane CD.

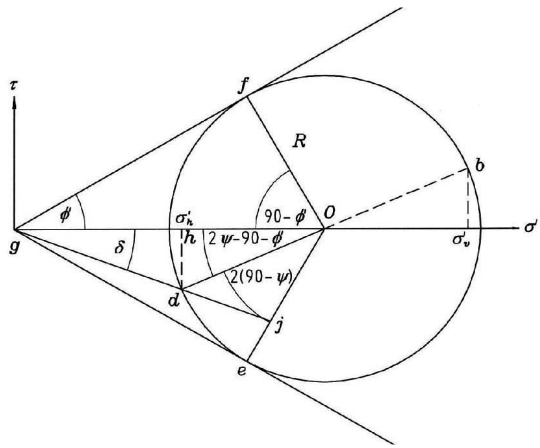


Figure A2. Mohr's circle for the critical wedge

The angle between planes EC and ED is (90-ψ), so this transformation of axes requires a rotation on the Mohr's circle from point *e* to point *d* of

$$e\hat{o}d = 2(90-\psi) \qquad \text{(A1)}$$

Triangle Δgeo is right angled, hence

$$e\hat{o}g = 90-\phi' \qquad \text{(A2)}$$

From (A1) and (A2),

$$d\hat{o}h = 90-\phi'-2(90-\psi) = 2\psi-90-\phi' \qquad \text{(A3)}$$

The external angle of triangle Δgdo is therefore:

$$o\hat{d}j = 2\psi - 90 - \phi' + \delta \qquad \text{(A4)}$$

By considering triangles Δdjo and Δgjo and using the sine rule,

$$oj\sin(d\hat{j}o) = R\frac{\sin\delta}{\sin\phi'} = R\sin(2\psi-90-\phi'+\delta) \qquad \text{(A5)}$$

Leading to:

$$\frac{\sin\delta}{\sin\phi'} = \sin(2\psi-90-\phi'+\delta) \qquad \text{(A6)}$$

Rearranging (A6) gives

$$\psi = \frac{1}{2}\left[90+\phi'-\delta+\sin^{-1}\left(\frac{\sin\delta}{\sin\phi'}\right)\right] \qquad \text{(A7)}$$

Now consider the equilibrium of the wedge component triangles ΔEDC and ΔCDF as shown in Figure A3.

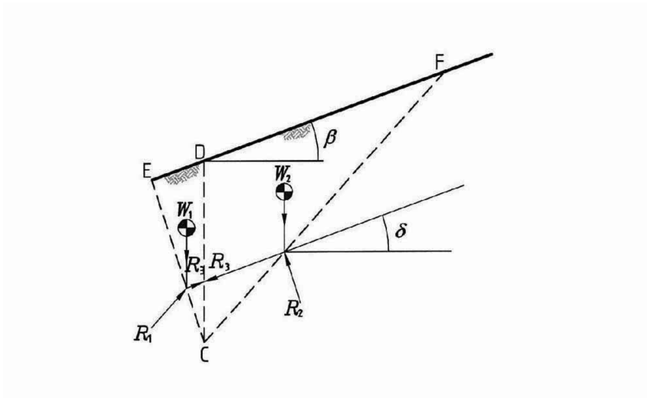


Figure A3. Equilibrium of wedge component triangles

The forces R_1 and R_2 act at the 1/3 points of CE and CF, because there is a linear stress distribution along these lines. These points are vertically below the centroids of triangles ΔEDC and ΔCDF. The force R_3 acts at the interface CD at an angle δ. For equilibrium the 3 forces for each triangle must intersect at a point. These intersection points must therefore be the 1/3 points on the boundaries CE and CF, as shown in Figure A3. Using similar triangles, the angle of the force R_3 must be identical to the slope of the backfill[1], or:

$$\delta = \beta \tag{A8}$$

From (A7) and (A8),

$$\psi = \frac{1}{2}\left[90 + \phi' - \beta + \sin^{-1}\left(\frac{\sin\beta}{\sin\phi'}\right)\right] \tag{A9}$$

By summing the angles around point O in Figure A2 to 360 degrees it can also be demonstrated that the angle ε is related to ψ by the expression in (A10):

$$\varepsilon = 90 + \phi' - \psi = \frac{1}{2}\left[90 + \phi' + \beta - \sin^{-1}\left(\frac{\sin\beta}{\sin\phi'}\right)\right] \tag{A10}$$

Knowing the critical value of ψ as given in (A9), the ratio of the horizontal to the vertical earth pressures may be determined from the Mohr's circle in Figure A2:

$$\frac{\sigma'_h}{\sigma'_v} = \frac{\dfrac{1}{\sin\phi'} - \sin(2\psi - \phi')}{\dfrac{1}{\sin\phi'} + \sin(2\psi - \phi')} = \frac{1 - \sin\phi'\sin(2\psi - \phi')}{1 + \sin\phi'\sin(2\psi - \phi')} \tag{A11}$$

[1] An alternative derivation of (A7) considers the triangles of forces for the component triangle ΔEDC and the full wedge ΔECF shown in Figure A3 and demonstrates that when $\delta=\beta$ the horizontal thrust is equal in each case; however the derivation is more complex and so not included here for space reasons.

However, while the ratio of stresses defined in (A11) could be thought of as an earth pressure coefficient, the values obtained from (A11) are not the same as K_a defined in the conventional way as in (A12), based on the total horizontal force H acting on a vertical plane of height h and assuming a vertical earth pressure of γz, where γ is the soil density and z is the distance below ground level.

$$K_a = \frac{H}{\frac{1}{2}\gamma h^2} \tag{A12}$$

H may be determined by considering the equilibrium of the wedge component triangle ΔCDF in Figure A3, from which:

$$H = \frac{W_2}{\tan\delta + \dfrac{1}{\tan(\varepsilon - \phi')}} \tag{A13}$$

The weight W_2 is calculated based on the area of the triangle ΔCDF:

$$W_2 = \frac{1}{2}\gamma h^2 \frac{\cos\varepsilon\cos\beta}{\sin(\varepsilon - \beta)} \tag{A14}$$

Combining (A8), (A12), (A13) and (A14) results in the following expression for K_a:

$$K_a = \frac{\cos\varepsilon\cos\beta}{\sin(\varepsilon - \beta)\left[\tan\beta + \dfrac{1}{\tan(\varepsilon - \phi')}\right]} \tag{A15}$$

A comparison of expressions (A11) and (A15) shows that they give almost identical values up to a slope angle β of about half ϕ', but as the slope approaches ϕ' the values diverge, with (A15) giving higher values. This difference is due to the way that K_a has been defined in (A12), which is convenient for design purposes, but this definition of K_a is not strictly the same as the ratio of horizontal and vertical pressures when the backfill is sloping.

The values for K_a presented in Table 4 of PD6694-1 have been calculated from (A10) and (A15) for various values of β and ϕ'.
Equation (A9) may be used to check whether $\psi > \alpha$. If this is not satisfied (i.e. the heel is short) then the derivation above is not correct; the angle δ will lie somewhere between 0 and β, and the thrust on the wall will need to be increased as described in PD6694-1.

Bridge Design to Eurocodes: UK Implementation
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DEVELOPMENT OF TRAFFIC SURCHARGE MODELS FOR HIGHWAY STRUCTURES

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Abstract
Models have been developed to represent the horizontal load surcharges on abutment walls, wing walls and other earth retaining structures due to traffic loads. These models have been developed based on an analysis of the global and local effects of the traffic loads in the UK National Annex to BS EN1991-2, and are different from the uniform pressure approach of BD37[1].
The recommended approach for abutments is based on the application of a horizontal uniform load together with knife-edged loads at the surface. For other structures such as wing walls a different is approach is needed, involving superposition of the effects of wheel loads. These models have been incorporated into PD6694-1[2] for structures subject to traffic loading and designed to BS EN1997-1.

Notation
All notation is based on the definitions of PD6694-1, BS EN 1991-2 and its National Annex.

Introduction
This paper describes the development of requirements as included in PD6694-1[2] for the modelling of horizontal surcharge effects caused by the vertical traffic loading applied to the carriageway behind abutments, wing walls, side walls and other parts of the bridge in contact with earth.

Before the introduction of Eurocodes, the standard approach for designing highway structures for traffic surcharge effects in the UK followed the requirements of BD37/01[1], which specified a vertical live load surcharge behind an abutment of 10kN/m² for HA loading and 20kN/m² for 45 units of HB loading (BD 37/01 **5.8.2**). This vertical load was typically converted into a horizontal earth pressure for design using an appropriate earth pressure coefficient, *K*. The validity of the BD37/01[1] surcharge loads is somewhat questionable. The 10kN/m² vertical surcharge for HA loading first appeared in BS153[3], when it was approximately equivalent to the uniformly distributed load (UDL) component of HA loading over a 4.5m loaded length. However, the localised effects that would normally have been modelled with the knife edged load (KEL) component of HA loading were not included, and the magnitude of the surcharge loading was not updated to align with subsequent increases in allowable traffic weights.

The 20kN/m² surcharge load that was intended to model 45 units of HB does not seem to be consistent with the magnitude of the HB load model (a single bogey for 45 units of HB had an average surface pressure of around 130kN/m², which is 6.5 times greater than the 20kN/m² surcharge load).

The uniform pressure method of BD37/01[1] also does not realistically represent the distribution of pressures on the wall due to vehicle loading; the pressures should be more concentrated towards the top of the wall (this will be demonstrated later in the paper, based on a variety of analytical methods). With the implementation of Eurocodes it was necessary to develop rational models for surcharge based on the traffic loading specified in the UK National Annex to BS EN 1991-2 and satisfying the requirements of BS EN 1997-1. The new surcharge models as stated in PD6694-1 and described in this paper were developed to properly account for surcharge effects, and are more realistic and also more onerous than the past practice as specified in BD37/01.

Traffic Load Models in BS EN 1991-2

The traffic loading for the carriageway behind abutments and wingwalls and other parts of structures in contact with the earth is covered by BS EN 1991-2, **4.9.1**, and defined in the UK National Annex to BS EN 1991-2, **NA.2.34**. For normal traffic, the load model in Figure 1 is used in place of Load Model 1 (in this paper for convenience we refer to the load model in Figure 1 as the "equivalent LM1 vehicle"). The SV and SOV load models (as defined in UK National Annex to BS EN 1991-2, **NA.2.16**) may also be required.

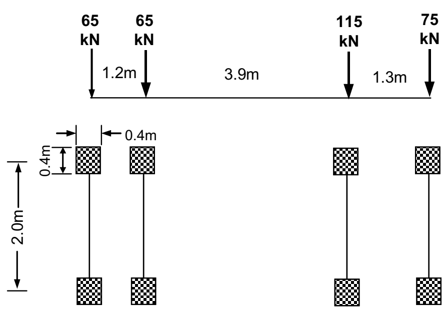


Figure 1. Load model for normal traffic (equivalent LM1 vehicle)

The axle loads in Figure 1 must be multiplied by an overload factor of 1.5 and a dynamic amplification factor (DAF) of 1.4, although for effects below the surface the National Annex allows the DAF for this vehicle and the SV and SOV vehicles to be reduced linearly to unity at a depth of 7m below the surface. (This slow rate of reduction is much more conservative than allowed in the UK assessment standard BA55[4] and the Canadian Highway Bridge Design Code CSA-S6[5] which both reduce the DAF to a minimum value at just 1.5m below the surface.) For vehicles in lanes other than lane 1, the loads should also be reduced by a lane factor as defined in BS EN 1991-2.

If the vehicle weights (including the overload factor and the DAF) are assumed to be uniformly distributed over the plan area of the vehicle (or axle group) the resulting average vertical pressures at surface level are as given in Table 1. These are considerably higher than the 10kN/m² for HA loading and 20kN/m² for HB loading specified in BD 37[1].

Average pressure under one equivalent LM1 vehicle	36 kN/m²
Average pressure under rear pair of equivalent LM1 axles	98 kN/m²
Average pressure under trailer of the SV196 vehicle	56 kN/m²
Average pressure under trailer of SOV vehicles	66 kN/m²

Table 1. Approximate average pressures for vehicle loads.

Surcharge Analysis Methods

There are a number of theoretical methods that can be used for the analysis of surcharge pressures. These methods vary in approach, from some that are based on elastic properties (Boussinesq[6]) to others that are based on slip failures in the soil (Coulomb wedge analysis[7], Williams and Waite[8]) and also including some empirical and semi-empirical methods (BD37[1], CP2[9]). Another family of methods is based on the approach of first modelling the distribution of vertical stresses in the soil adjacent to the wall (there are various ways to attempt this) and then multiplying these by an earth pressure coefficient, based on the theory of Rankine[10] to calculate horizontal earth pressures.

Unfortunately there is apparently no single method that alone provides a high level of confidence in modelling the soil and its interaction with the structure accounting for its non-linear behaviour. For this reason and also to explore the sensitivity of the results to the method used, a variety of methods were considered in developing the simplified surcharge models.

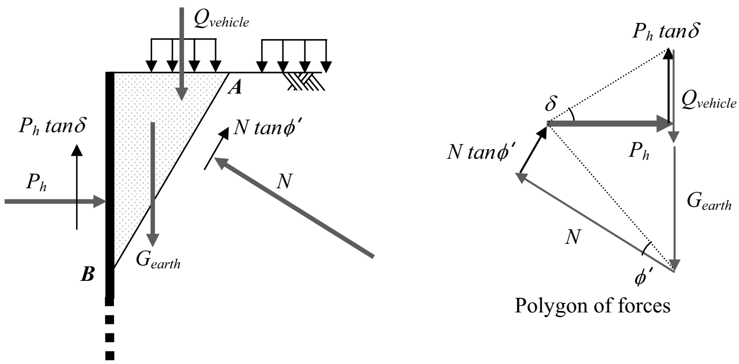
This paper focuses on the following analytical methods, illustrated in Figure 2:

- Coulomb wedge analysis[7] to iteratively determine critical horizontal thrust
- Rankine method[10] based on elastic vertical pressures from Boussinesq analysis[6]
- CIRIA C580[11] method based on Williams and Waite[8].

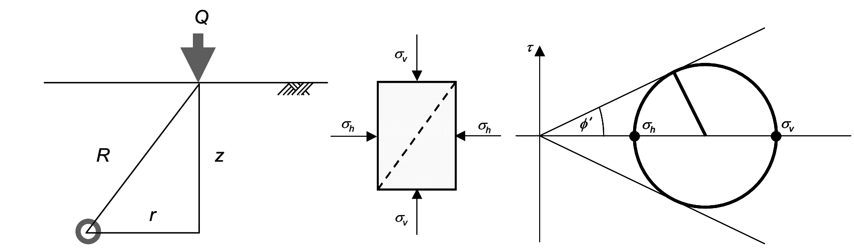
Development of a Surcharge Model for Abutments

Global effects

The theoretical effects of the equivalent LM1 vehicle, the SV vehicles and the SOV vehicles have been considered using the analytical methods in Figure 2, assuming an abutment wall crossing the carriageway. The analysis generally predicts a significant concentration of pressure at the top of the wall, as illustrated in Figure 3 for the equivalent LM1 model and the Rankine[10] /Vertical Boussinesq[6] method.



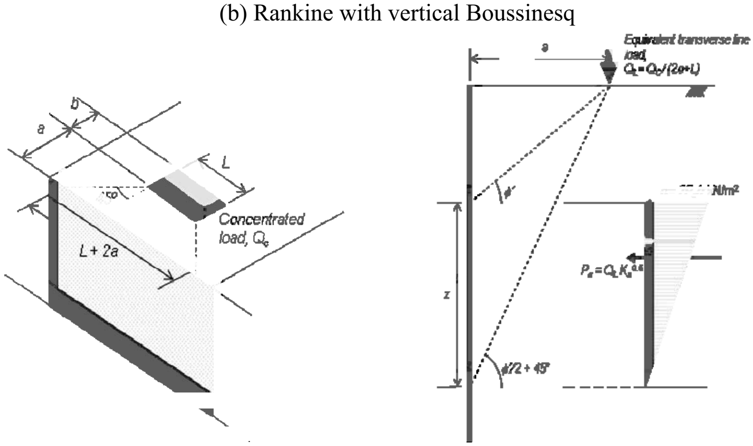
(a) Coulomb wedge analysis



$$\sigma_v = \frac{3Qz^3}{2\pi R^5}$$

$\sigma_h = K_a\sigma_v$ *where* $K_a = (1 - \sin\phi')/(1 + \sin\phi')$.

(b) Rankine with vertical Boussinesq



(c) CIRIA C580

Figure 2. Surcharge analysis methods

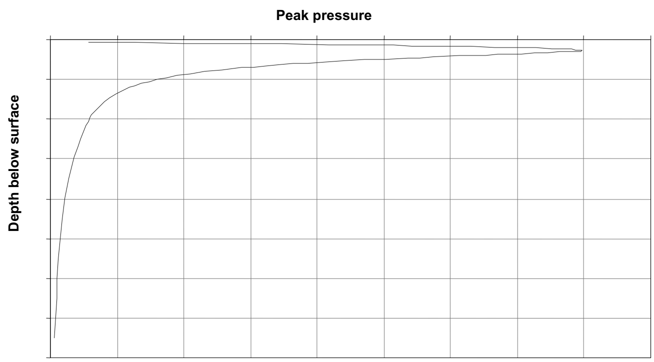


Figure 3. Distribution of pressures for normal (equivalent LM1) loading (based on K_a applied to vertical Boussinesq pressures)

A simple and convenient method for comparing the global effects of the various methods has been used, where envelopes of the total shear force caused by one lane of traffic loading have been plotted at intervals down a simple cantilever wall. The results of this comparison are illustrated in Figure 4 for a ϕ_d' angle of 33 degrees (K_a=0.3). The diagram for SOV vehicles is based on the most onerous SOV vehicle configuration, although similar results are obtained for shorter SOV vehicles.

Figure 4 includes the effects of reducing the DAF with depth. For the Rankine/Boussinesq and C580 methods, the pressures were calculated based on the DAF at each depth and then integrated to find the shear force. For the Coulomb wedge method[7] this approach was not directly possible, and so the live loads were multiplied by the DAF, with the DAF based on the depth a third of the way down the wedge being considered. For comparative purposes, the effects of the self weight of the earth have subsequently been subtracted from the Coulomb wedge results.

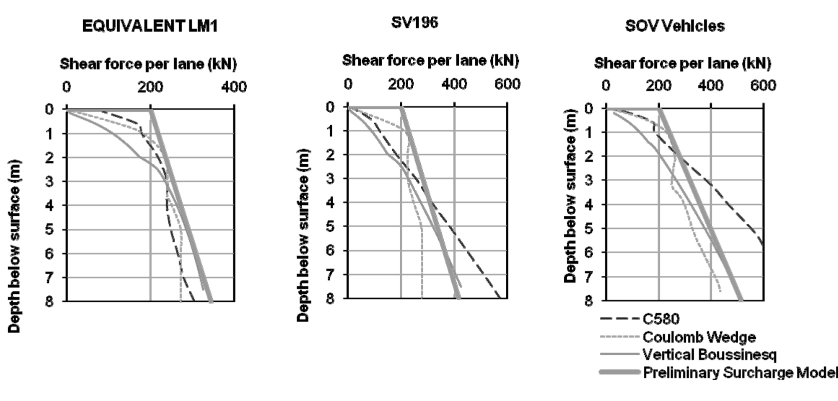


Figure 4. Comparison of shear forces caused by surcharge loading

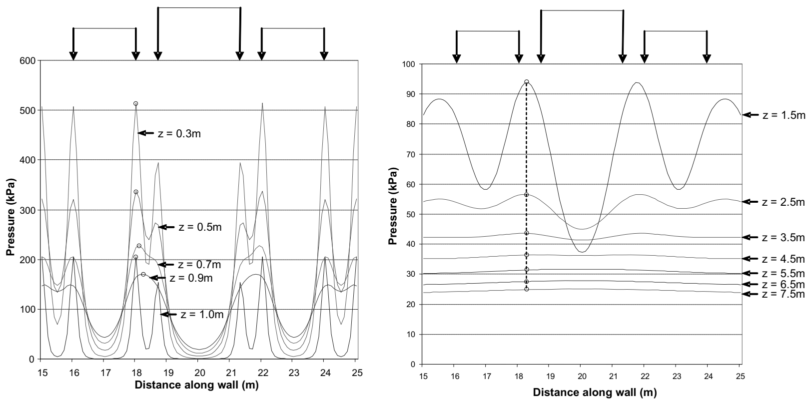
As seen in Figure 4, the global effects for a lane of vehicle loading according to these models can be reasonably modelled by a shear force of 200kN (≈660K_a kN) at the top of the wall increasing linearly with depth. Hence the preliminary form of the surcharge model for global effects was developed in the form of a horizontal knife edged load (KEL) at the top of the wall of 660K_d kN combined with a horizontal uniformly distributed load (UDL) with a magnitude of 20K_d kPa for normal loading, 30K_d kPa for SV196 and 45K_d kPa for SOV loading, where K_d is the design value of K_a for flexible walls or K_0 for rigid walls, based on ϕ_d'. The effects of SV100 were slightly less onerous but similar to SV196 (the SV 100 vehicle is identical to part of the SV196 vehicle) and SV80 loading were found to be slightly less onerous than normal loading. For design it was considered reasonable to have three levels of loading, corresponding to (i) normal loading or SV80 loading, (ii) SV196 or SV100 loading, and (iii) all SOV vehicles.

Figure 4 shows that the C580 method did predict higher pressures than the preliminary model for the SV and SOV vehicles at greater depths, however it was considered that the C580 method was probably more appropriate for small concentrated loads near the wall and could be rather conservative when applied to large vehicles with many axles extending far from the wall. (The other methods considered were generally insensitive to loads further than about H from the wall.)

Local concentrations of pressure and short walls

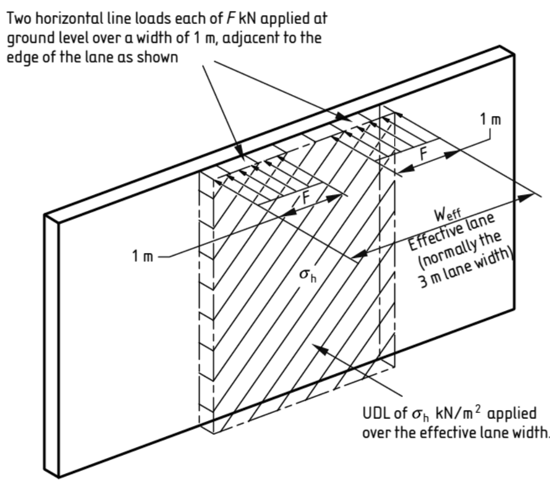
Aside from the global effects of each lane of traffic loading, there can be some intense peak local pressures associated with wheel loads. These local pressures may be particularly important for segmental structures where high localised earth pressures could be applied to a single precast unit. The Rankine method with vertical Boussinesq pressures was used to investigate the distribution of local peak pressures. These are illustrated in Figure 5 for the example of a SV 196 vehicle with equivalent LM1 vehicles in adjacent lanes.

As seen in Figure 5, the local peaks in pressure are mainly confined to the top few metres of soil. This effect may conveniently be modelled using an adjustment to the application of the KEL component of the model that was previously described for global effects. From an analysis of the effects on the most critical metre strip for a variety of load configurations it was found that the model shown in Figure 6 could be used to determine both global and local effects. By applying the KEL component over two 1m-wide strips at the edges of the lane, the effects of the local pressures were adequately modelled.



(a) Pressures in top 1m of soil **(b) Pressures from 1.5m to 7.5m depth**

Figure 5. Horizontal pressures for SV196 flanked by equivalent LM1 vehicles using the Rankine / Boussinesq method



	F	σ_h
Normal Loading For normal loading in lanes other than lane 1 these loads may be reduced using the lane factor in EN1991-2	$330K_d$ kN/m	$20K_d$ kPa For lane widths W_{eff} narrower than 3m this should by increased by a factor $3m/W_{eff}$.
SV 100 or SV196 Loading	$330K_d$ kN/m	$30K_d$ kPa
SOV Loading	$330K_d$ kN/m	$45K_d$ kPa

Figure 6. Horizontal surcharge loading model (characteristic values)

Transverse structural distribution

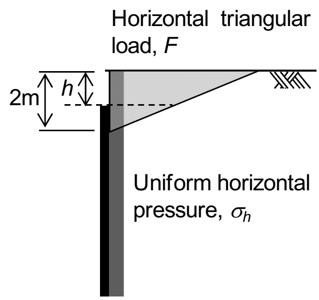
The loads in Figure 6 may be used to model the effects of surcharge loading accounting for the distribution of pressures in the soil. However, for stiff laterally continuous walls the structure itself will have a further influence in distributing the effects of localised pressure concentrations. While the loads in Figure 6 would be appropriate for a complete model of the structure that accounts for the structural distribution, they may be rather conservative for a unit strip approach that neglects the structural distribution. Such an approach is a popular calculation design method for some structure types, and so a further adjustment factor has been developed to account for transverse distribution to allow the model to be used in conjunction with a unit strip approach without undue conservatism.

Finite element models were generated to simulate concrete walls of various wall heights and thicknesses. The models were loaded with the pressures generated by the Rankine / Boussinesq method. In general, the results of these analyses indicated that the moments and shears in the walls were almost uniformly distributed across the width of the walls (except at the very top), and that the localised effects of pressure concentrations did not have a significant effect on the distribution of structural effects.

By comparing the results of the analyses for walls of various heights, the degree of structural distribution was investigated. This investigation suggests that for a metre strip analysis of a vertically spanning transversely stiff wall, the moments and shears caused by the KEL component F of the loads in Figure 6 may be reduced by the empirically determined factor $(1+0.5z)/(1+z)$, where z is the depth below the top of the wall. However, this factor should not be taken as less than 2/3 (which corresponds to the KEL component of total width 2m becoming uniformly distributed over the 3m wide lane). The effects of the UDL component σ_h should not be reduced.

Buried structures

The use of a horizontal KEL at surface level to represent the concentration of loading near the surface is an appropriate simplification for structures that are not buried. However, the influence of the concentration of pressures may also affect buried structures where the top of the structure has less than 2m depth of fill. A reduction factor has been developed to be applied to the KEL component F, with the form of the factor based on approximating the pressures in the top 2m to a triangular distribution, as shown in Figure 7. The triangular distribution is consistent with the parabolic variation in shear force towards the top of the wall that can be seen in Figure 4. Hence for structures where the top of the structure is buried but at a depth h (in metres) less than 2m below ground level, F is reduced by a factor $\left(1-\frac{h}{2}\right)^2$ and applied at the top of the structure. For structures buried deeper than 2m, the KEL component F is not applied. The UDL component σ_h is unaffected.



Proportion of triangular load applied to wall = $\left(1-\frac{h}{2}\right)^2$

Figure 7. Adjustment for buried structures

Development of a Surcharge Model for Wing Walls and Other Earth Retaining Structures

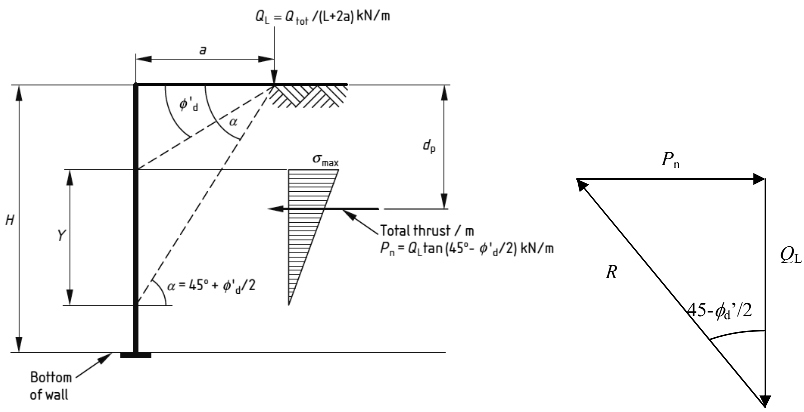
The effect of live load surcharge on wing walls is different from that on abutments walls. The lanes are not perpendicular to the wall and so the relative position and orientation of the vehicle loads are different from abutment loading. Wing walls may run parallel to the carriageway or can be set at an angle to the carriageway, and in many cases will be remote from the edge of the carriageway. The geometry of wing walls is also often non-uniform. The method in PD6694-1[2] for wing walls and other earth retaining structures is based on the method of C580[8], and requires superposition of the effect of patch loads associated with traffic loading.

For loads that are reasonably close to the wall as illustrated in Figure 8 (a), i.e. where $a < H \tan\left(45 + \frac{\phi'_d}{2}\right)$ the horizontal thrust is determined from the triangle of forces as shown in Figure 8(b) as:

$$P_n = Q_L \tan(\alpha - \phi'_d) = Q_L \tan\left(45 - \frac{\phi'_d}{2}\right) \tag{1}$$

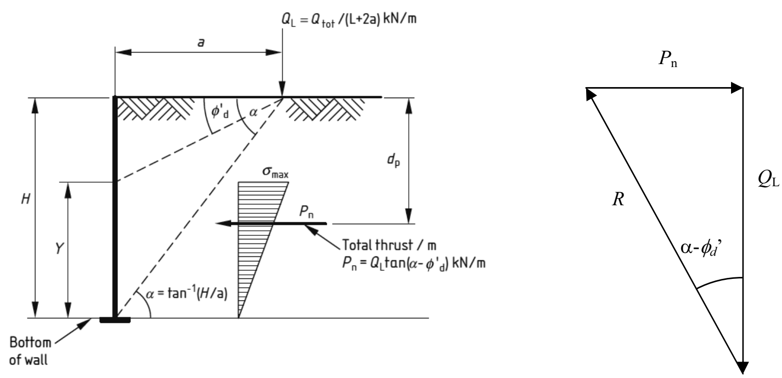
where α=45+ϕ_d'/2. The form of equation (1) is trigonometrically identical to the C580 expression, given in (2):

$$P_n = Q_L \sqrt{\frac{1-\sin\phi'_d}{1+\sin\phi'_d}} = Q_L \sqrt{K_a} \tag{2}$$



(a) (b)

$$a < H \tan\left(45 + \frac{\phi'_d}{2}\right)$$



(c) (d)

$$a > H \tan\left(45 + \frac{\phi'_d}{2}\right)$$

Figure 8. Method for wing walls

For loads that are further from the wall so that $a > H \tan\left(45 + \frac{\phi'_d}{2}\right)$ CIRIA C580 was not explicit in how to apply the method. If the model of Figure 8 (a) were used then the active wedge of soil would extend below the bottom of the wall, and the boundary condition at the vertical interface would have changed. In developing the PD6694-1 methodology, two alternative approaches were initially considered. The first was to use the method of Figure 8(a) but disregarding the pressures below the bottom of the wall. The second method (which seemed more rational) was to adjust the method with a reduced value of α to fit within the wall geometry as shown in Figure 8(c) and (d). This approach gives a thrust of:

$$P_n = Q_L \tan(\alpha - \phi'_d) \tag{3}$$

where

$$\tan\alpha = \frac{H}{a} \tag{4}$$

A comparison was carried out for the method in Figure 8(c) and the method in Figure 8(a) but disregarding the pressures below the bottom of the wall. This comparison yielded almost identical bending moments about the base of the wall and comparable shear forces for the two methods. The recommendation in PD6694-1 was therefore to have a minimum value of α as illustrated in Figure 8(c), which seems to be a rational and sensible solution. A classical Coulomb wedge analysis would indicate that axles further than a distance of approximately H from the wall do not influence the horizontal thrust, and the methods of Figure 8 (where there is no such limit) are on the conservative side for loads remote from the wall.

The practical problem with using the methods in Figure 8 is that the effect of every wheel of every vehicle apparently needs to be superimposed, which can be laborious. A simplification may be made for analysing global effects where the wall is parallel to the carriageway, by considering the effect of each line of wheels of a vehicle or a convoy of vehicles and then summing these effects together with those for any vehicles in other lanes.
For the case where the wall is longer than $L+2a$, where L is the length of the vehicle and a is the distance from the wall to the line of wheels, the horizontal thrust associated with each line of wheels may be modelled by taking the sum of the wheel loads in the line of wheels (ΣW) and multiplying by $\tan(\alpha - \phi'_d)$ using the method of Figure 8. The *average* thrust per metre of wall associated with the line of wheels is therefore

$$P_{n,ave} = \frac{\sum W}{L_{wall}} \tan(\alpha - \phi'_d) \tag{5}$$

However, where the wall length does not exceed $L+2a$ (or if there is a convoy of vehicles) then it is necessary to superpose the effects of wheels and to determine the critical vehicle position to give the maximum thrust. An alternative approach would be to develop tables giving the worst average thrust per metre for walls of various lengths and for various distances (a) from the line of wheels to the wall, for each vehicle configuration required.

The UK National Annex to BS EN 1991-2 allows the dynamic amplification factor (DAF) for both vertical and horizontal effects to be linearly reduced according to the depth below the ground surface. This means that when applying the model of Figure 8 the pressures at each depth are subsequently multiplied by a DAF that reduces with depth, resulting in a total pressure distribution that is parabolic rather than triangular. However, an acceptable degree of accuracy is generally obtained by using a constant DAF based on the depth d_P of the centroid of the triangular pressure diagram in Figure 8.

Conclusions

Methods for analysing the effects of live load surcharges on abutments, wing walls and other earth retaining structures have been developed corresponding to the load models in the UK

National Annex to BS EN1991-2. The surcharge models have been incorporated into PD6694-1.

The model for surcharge on abutments comprises UDL and KEL components to be applied to the abutment, and is appropriate for a variety of structure types including segmental structures where local pressure concentrations may be critical. To allow more economical unit-strip design of structures that are able to distribute loads transversely, a reduction factor has been derived that is a simple function of the height of the wall. This factor may then be applied to the KEL component. The abutment surcharge model is also appropriate for buried structures, with an adjustment necessary for structures with a depth of fill less than 2m.

Recommendations have also been made for modelling surcharge effects on wing walls and other earth retaining structures, and guidance has been developed to facilitate the modelling of these effects.

Acknowledgements

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DEVELOPMENTS IN INTEGRAL BRIDGE DESIGN

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Abstract
The Eurocodes do not cover important aspects of the design of integral bridges, including the behaviour of soil behind bridge abutments that is subject to repeat thermal movement. PD 6694-1[1] therefore contains recommendations and guidance for the design of integral bridges. These update the UK recommendations for integral bridge design contained in BA 42/96[2] in the light of recent research and to align them with the Eurocode design philosophy. This paper explains their background.

Notation
In this paper the same notation is used as in the Eurocodes and PD 6694-1. Other symbols are defined within the clause in which they occur. The Clause numbers used in the headings of this paper are the Clause numbers in the PD to which the text refers. References to PD clauses are given in bold text.

Introduction
The original recommendations for the design of integral bridges promulgated by the Highways Agency and Devolved Administrations were contained in BA 42/96[2]. This document has been used for over 13 years, during which time it has been found that there are a number of anomalies and potential ambiguities in the text and some of the clauses have the potential to be misinterpreted. Moreover, during this period a considerable amount of research has been carried out on integral bridge behaviour leading to a better understanding of the response of backfill to thermal cycling.

The Eurocodes do not cover important aspects of the design of integral bridges, and PD 6694-1[1] (hereafter referred to as "the PD") therefore contains recommendations and guidance for the design of integral bridges. These update BA 42/96. This paper explains the background to the content of PD 6694-1 on the design of integral bridges, and broadly follows the structure of the PD. Comparisons are made between the recommendations in PD 6694-1 and BA42/96. While some important changes between the two documents have been made to align the recommendations with Eurocode philosophy, many of the changes have been made in the light of recent research work or to clarify clauses which caused confusion in BA 42/96.

Background to other aspects of PD 6694-1 is provided by Denton et al[3].

Restrictions on limit equilibrium design (9.2.1)

The PD gives options for designing integral bridges using either a limit equilibrium approach or a soil structure interaction analysis. There are, however, some restrictions on using the limit equilibrium design method included in the PD. These restrictions have been modified from BA 42/96 and are summarised below.

Length of bridge (9.2.1a)

As the enhanced K^* pressures behind an integral abutment are a function of the movement of the end of the deck rather than the length of the deck, the length of bridges for which the limit equilibrium analysis methods described in the PD are applicable is defined in terms of the characteristic thermal movement of the end of the deck of the bridge rather than the overall length of the bridge as was done in BA 42/96.

Skew (9.2.1b)

The limit on a 30^{o} skew is unaltered.

Types of structures for which limit equilibrium methods are not considered to be appropriate (9.2.1c)

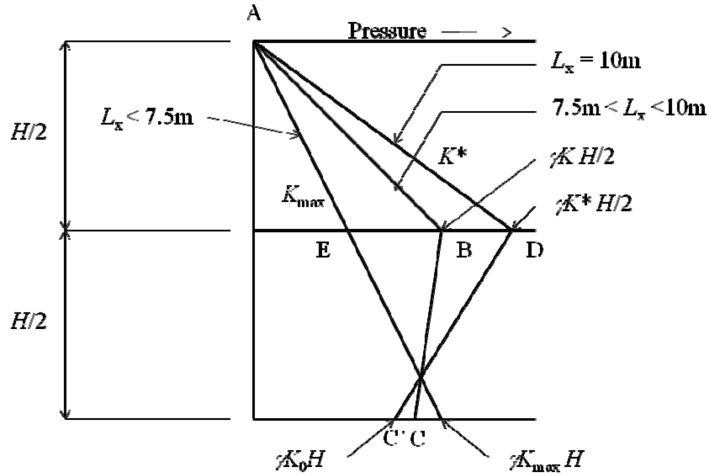
As a result of research into soil-structure interaction design methods carried out in recent years (see below), it was found that the behaviour of embedded walls and full height frame abutments on a single row of piles predicted by limit equilibrium methods could not be reconciled with results obtained using soil-structure interaction methods. For these types of abutment a soil structure interaction analysis is now recommended.

Soil structure interaction (9.2.2)

The PD is not prescriptive on the form of soil structure analysis undertaken, but does state criteria that must be fulfilled (see also **9.4.5**) and provides a method that satisfies these criteria (see **Annex A**). Importantly, **9.2.2** explains that the soil structure interaction approach must have been calibrated against comparable experience, laboratory modelling and/or case history data experience. For further background to the recommendations relating to soil-structure interaction analysis, reference should be made to the text relating to **9.4.5** and in particular **Annex A** of the PD included at the end of this paper.

Short Bridges (9.2.3)

In past design practice in accordance with the Design Manual for Roads and Bridges (DMRB)[4], buried integral structures up to 15m in length (*i.e.* an expansion length of 7.5m) were designed in accordance with BD 31[5]. Bridges longer than that were required to be designed using BA 42/96. This resulted in a step-change between the designs of structures which were just shorter than and just longer than 15m. The PD allows for a transition in the design pressures applied to bridges with expansion lengths of between 7.5m and 10m **(9.2.3)**. Figure 1 illustrates a method for applying this transition for one design case, with partial factors omitted for clarity.



Notes:

(a) A structure has an expansion length L of between 7.5m and 10m

(b) K^* is the relevant design value of K^* for an integral bridge with L_x =10m

(c) K_{max} is the design value of the pressure coefficient of a buried structure wall given in **Annex B Table B.1** of the PD

(d) ABC is the pressure diagram for the structure with B located such that: EB/ED = (L_x-7.5)/2.5

(e) K_{max}+ (L_x - 7.5)(K^*- K_{max})/2.5 is the interpolated value of K^* for the structure.

(f) Position C is plotted strictly in accordance with the recommendations of the PD although it would be more logical to move C to C' so that the pressure at H was at rest pressure.

Figure 1. Diagram showing how the pressure distribution on a full height abutment may be considered to vary when the expansion length of the deck lies between 7.5m and 10.0m

Types of bridges for integral construction (9.3)

The types of bridges explicitly considered for integral construction have been increased to include bridges on flexible supports such as piles in sleeves (**Figures 5(h) - 5(j)**).

The bridges fall into two categories: full height frame abutments in which the movement is accommodated by rotation or flexure of the abutment wall; and, end screen abutments which include bank pad abutments, flexible support abutments and semi integral abutments in which thermal movement is accommodated by the end of deck translating in and out of the fill.

Earth pressures behind integral abutments and end screen walls (9.4)

Background and research review

In 2005 the Highways Agency, commissioned a series of studies by Arup and Parsons Brinckerhoff to ensure that integral bridge design guidance reflected the latest understanding. This work included a full review of existing data, back-analysis of measured performance, a review of existing standards, and the development of recommendations for new design methods[11,12,13]. The findings of these studies have been incorporated into PD 6694-1.

The behaviour of soils behind integral bridge abutments has been examined by a number of researchers, using a diverse set of 'Laboratory models', including England et al[6], Springman and Norrish[7], Tapper and Lehane[8], Clayton et al[9], and Tan and Lehane[10]. The findings of these studies demonstrated, in a consistent and quantifiable form, the response of granular soils to repeated cycles of strain. England et al[6], for example, demonstrates that as a rigid wall rotates about a hinge at its base, soil pressures cycle between active and mobilised passive pressures. Each repeated cycle results in an increase in soil pressure, however the increase tails off after 100 to 200 cycles (see Figure 1).

The resulting earth pressure increases almost linearly with depth over the top half of the soil before falling back towards the K_o value at the hinge location. The form of pressure distribution suggests that the upper soil pressures are governed by the mobilised passive resistance of the soil (which can be modelled using a quasi-passive limit), and that the pressures in the lower half of the soil are principally generated by the soil stiffness as the wall pushes into the soil.

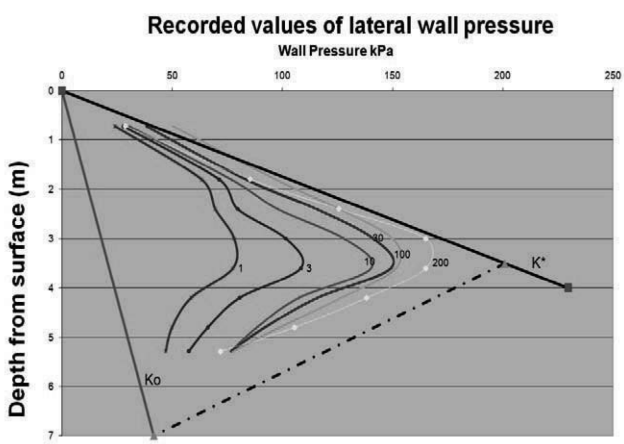


Figure 1: Comparison of measured (England et al[6]) and calculated (PD 6694-1) soil pressures

The mobilisation of passive resistance during the rotation of a rigid vertical plate, and the identification of the relationship between soil strain and the resulting ratio of horizontal and vertical stresses has been investigated by Terzaghi[14] and later work by Hambly and Burland[15].

The findings of the research on integral bridge abutment behaviour indicate that the mobilised passive resistance due to repeated cycling of a rigid wall (K* pressures) is significantly greater than that reported by Terzaghi and Hambly and Burland. There are two principal reasons for this: firstly, the effect of moving a full thermal cycle from an active position results in significantly higher pressures than the corresponding half cycle movement from an at rest position; and secondly, the mobilised passive resistance of the soil increases with each repeated cycle of movement.

One further issue is noteworthy in connection with the back analysis of the measured performance of the laboratory models[6,7,8,10], as investigated by Arup[13]. This is that the soil parameters quoted were not consistent: φ' was never given based on plane strain testing, rather either triaxial φ'_{max} or φ'_{cv} was quoted.

In order to enable a proper comparison of the different scale model results to be undertaken, it was necessary for a consistent φ' value to be used. In the back analysis, this was done by establishing a value of final triaxial φ'_{max} for all the tests (here denoted $\varphi'_{\text{max triaxial}}$). Table 1 provides a summary of how this was done. The following equations given by Bolton[16] and Clayton et al[9] respectively were used:

$$\varphi'_{\text{max triaxial}} = \varphi'_{cv} + 3\,(D_r(10 - \ln\rho') - 1) \qquad [1]$$

$$\text{Final } \varphi'_{\text{max triaxial}} = \text{Initial } \varphi'_{\text{max triaxial}} + ((0.9 - D_r)/0.1) \qquad [2]$$

where ρ' is the mean effective stress and D_r is the relative density.

	Initial Dr	Final Dr	φ'_{cv}	Initial $\varphi'_{\text{max triaxial}}$	Final $\varphi'_{\text{max triaxial}}$	Comment
Springman et al [7]	0.83	0.95	32.0		47.5	Final $\varphi'_{\text{max triaxial}}$ derived from quoted φ'_{cv} values using eq. [1]
England et al [6]	0.94	0.95	29.0	42.2	42.2	$\varphi'_{\text{max triaxial}}$ implicit in K_p of 12 quoted by England.
Tapper and Lehane [8]	0.50	0.95	32.0	36.0	40.0	Final $\varphi'_{\text{max triaxial}}$ derived from quoted initial $\varphi'_{\text{max triaxial}}$ values using eq. [2]
Tan and Lehane [10]	0.88	0.95	32.0		48.5	Final $\varphi'_{\text{max triaxial}}$ derived from quoted φ'_{cv} values using eq. [1]

Table 1: Derivation of Consistent Soil Parameters

Values of $K_{p;t}$ (9.4.1)
The values of $K_{p;t}$ for unfavourable passive pressure given in **Table 8** in the PD are based on $\delta/\varphi'_d = 0.5$ where $\tan\varphi'_d = \tan\varphi'_k/\gamma_M*$ and γ_M* is the reciprocal of γ_M (see notes under **Table A.NA.2** and **A.NA.4** in the National Annex to EN 1997-1). They were determined using the using the design values of the triaxial φ'.

The PD emphasises that it is important that the values of $K_{p;t}$ used should be based on triaxial φ', denoted φ'_{triax} This was done so that it is consistent with the way in which the equations for $K*$ were established (see above). The subscript 't' is used in $K_{p;t}$ to highlight that triaxial φ' is used, in contrast to value of φ' that would generally be used in conjunction with BS EN 1997-1:2004+A1, **Annex C**. Guidance is provided in **9.10.1** on the estimation of φ'_{triax} when triaxial test results are not available.

The design value of the movement range, d_d (9.4.2)
The characteristic value of the thermal movement range, d_k, is calculated from the minimum and maximum characteristic effective bridge temperatures determined in accordance with BS EN 1991-1-5.

In accordance with BS EN1990, a partial factor, γ_Q, and ψ factors are applied to thermal actions to determine the design value that is used in the various combinations of actions that need to be considered in the design. For the purposes of determining $K*$ pressures applied to integral bridge abutments, it is effectively considered that there is no thermal action at the mean characteristic temperature $(T_{e.max}+T_{e;min})/2$. Thus, the characteristic thermal expansion from the mean temperature position to the characteristic maximum temperature position is $0.5d_k$. Applying γ and ψ to this action gives a design expansion of $0.5d_k\psi\gamma_Q$.

Strictly the earth pressure applied to the abutment when an integral bridge expands will be affected by the maximum contraction that has occurred in that cycle of movement. However, taking account of this in the design would be cumbersome and is not considered necessary. Instead, the design value of the total thermal movement, d_d, is taken from the characteristic minimum temperature position, *i.e.* it is given by $0.5d_k + 0.5d_k\psi\gamma_Q = 0.5d_k(1+\psi\gamma_Q)$. The PD requires the $K*$ pressures be based on this movement as described below.

Abutments accommodating thermal movement by rotation and/or flexure (9.4.3)
There are three major changes to the $K*$ equations compared with those used in BA42/96, these concern: differentiation by structure type; the soil 'strain' parameter (d/H); and, the effect of a rigid boundary at the level about which the wall rotates.

Differentiation by structure type
BA 42/96 provided two equations for $K*$ for full height abutments:

(i) $K* = (d/0.05H)^{0.4} K_p$ for portal frame / full height embedded wall abutments; and
(ii) $K* = K_o + (d/0.03H)^{0.6} K_p$ for portal frame bridges hinged at the base of their legs.

As reported by Arup[11] the first equation is likely to be a recasting by Hambly of his earlier work and is no longer considered to realistically reflect the behaviour of integral bridges. This first equation has not been retained in PD 6694-1.

The PD uses a single equation for all abutments that accommodate thermal movement by rotation and/or flexure, based on the second equation above, that was originally derived from England et al[6], but modified for the reasons described below to give:

$$K^*_{d} = K_o + (Cd'_d/\mathrm{H})^{0.6}\, K_{p;t}$$

Treatment of the soil 'strain' parameter (d/H)

In BA42/96 the soil 'strain' was defined in terms of d/H. A back analysis[12] of the laboratory modelling carried out by Springman and Norrish[7] provided a direct comparison between cases of pure rotation and flexural displacement, and demonstrated that it is more reasonable to specify the 'strain' in terms of d_d'/H where d_d' is the movement of the wall at $H/2$ when the end of the deck moves a distance d_d.

As illustrated in Figure 2, the ratio d'_d/d_d varies depending on the boundary conditions at the top and bottom of the abutment. It is also affected by the stiffness of the wall and the magnitude of the earth pressure applied to it. For the case of a stiff wall hinged at the top and bottom (or fixed at the top and bottom), d_d' is about $0.5d_d$, and the term ($d/0.03H$) = ($33d/H$) used in the equation for K^* in BA42/96 becomes ($66d'_d$/H). As discussed below, $C = 66$ is one if the values used in the equation in **9.4.3** in the PD.

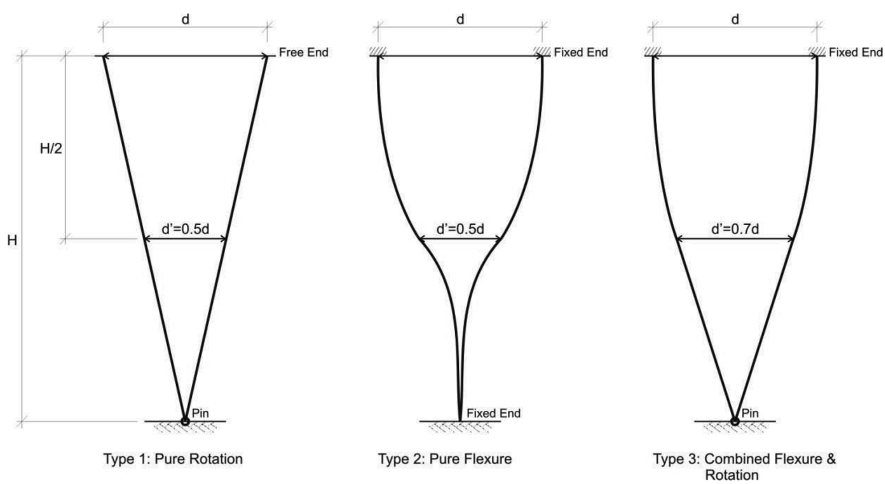


Figure 2: Comparison of different abutment movements

Rigid boundary effect

Utilising $K_{p;t}$ values derived using a consistent φ' value (see Table 1) allowed experimental results to be normalised and presented as shown in Figure 3. This demonstrated a clear difference between the findings of:

- England et al[6] and Tapper and Lehane[8] which used a rigid boundary to the soils at hinge level; and,
- Springman et al[7] and Tan and Lehane[9] which used no rigid boundary at the hinge location.

The removal of the rigid boundary at hinge level is believed to be significant as it allows soil particle rearrangements under repetitive loading to take place around and below hinge level resulting in a dissipation of the stresses in the soil.

From Figure 3, it can be seen that for the tests on models with a rigid boundary below the foundations a value of constant $C = 66$ in the equation in **9.4.3** provides a good fit to the experimental results. As discussed above, this value of C is equivalent to the BA 42/96 expression. However, for abutments without a rigid boundary (*i.e.* for flexible foundations) it is reasonable to reduce the constant C from 66 to 20. This corresponds to a reduction in K^*_d to about half that from BA42/96.

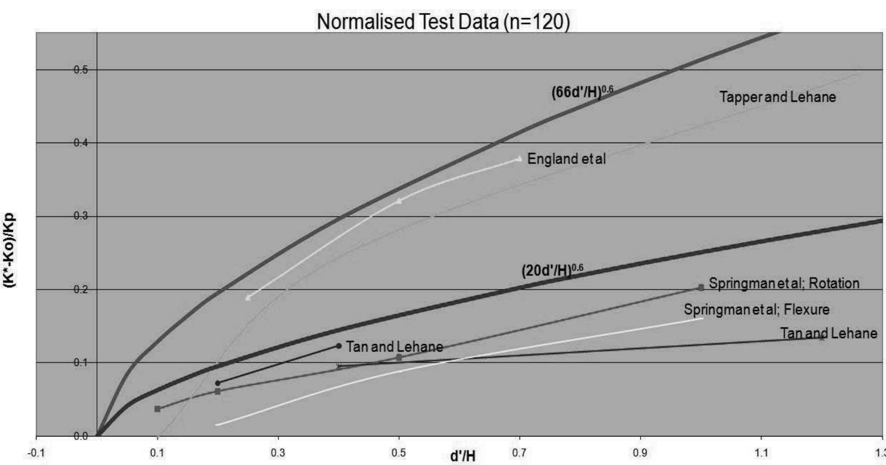


Figure 3: Normalised laboratory modelling results compared to PD 6694-1 *K equations**

Horizontal earth pressure on end screen abutments (9.4.4)

The equation for the K^* pressure on end screen abutments has not significantly changed as the rotation of the end screens is considered to be small and d_d' is therefore approximately equal to d_d.

Vertical distribution of *K** pressure. (9.4.3(1) and (2) and Figure 6)

The distribution diagram for K^* pressure has been altered. In BA 42/96 the K^* pressure increased linearly from zero at ground level to $\gamma K^* H/2$ at a depth of $H/2$ and thereafter remained constant down to the depth where it equalled the at rest pressure. In the PD the earth pressure below $H/2$ reduces linearly from $\gamma K^* H/2$ to $\gamma K_0 H$ between $H/2$ and H, the level at which no significant movement occurs (see **Figure 5(c)**). This is more logical because increases in earth pressure above K_0 would not be expected at H, and is consistent with experimental findings (see Figure 1).

Observations on maximum earth pressure when $\psi\gamma$ is not equal to 1 using *K** Equations in 9.4.3 and 9.4.4

The experimental work on which the K^* equations in **9.4.3** and **9.4.4** is based used repeated movement of constant magnitude over many cycles. In assessing the effects of repeated thermal cycles on the backfill of real integral bridges it was judged reasonable to consider the fundamental effects of strain ratcheting as being the same as those that would result from repeat thermal cycles with constant magnitude equal to the characteristic movement range, $d_{k.}$ Such an approach is essentially consistent with BA42/96.

However, for designs using the Eurocode philosophy it is necessary to determine not only the earth pressure that would arise due to the characteristic movement range, but also the pressures which occur with occasional movements d_d that may be greater or less than d_k, as described in the last paragraph. For an occasional movement of d_d greater than d_k (such as would occur at ULS with $\psi\gamma$ greater than 1) the resulting pressure will be less than the pressure based on repeated cycles of d_d but larger than the pressure based solely on repeated cycles of d_k.

In the extreme case with $\psi = 1$ and $\gamma_{Q;thermal} = 1.55$, the difference in the value of the $K_{p;t}$ term of the K^* equations determined using d_k and d_d is less than 16% (*i.e.* $(0.5(1+1.55))^n$, where n is 0.6 in **9.3.4** and 0.4 in **9.4.4**). For an occasional movement of d_d such as would occur at ULS, the increase in pressure will in reality be less than 16%. However, in the PD, the pressure due to an occasional movement of d_d is taken as the same as the pressure due to frequent cycling to d_d. Whilst this is not physically justifiable, it is very convenient and the degree of conservatism in making this assumption is a fraction of 16% and is considered insignificant in comparison to the approximations inherent in the K^* equations. Furthermore, it was recognised that a more conservative assessment than that described above for the minimum temperature in the cycle could have been made, which would effectively offsets the conservatism in the direct use of d_d in the K* equations.

A similar argument can be developed in relation to the situations where $\psi\gamma_Q$ is less than 1. Although in this situation actual pressure will be higher than pressure based on the K^* equations, the maximum difference is again only a few per cent.

9.4.5 Soil-structure interaction

The PD explains that for full height integral abutments founded on a single row of vertical piles and integral embedded wall abutments, the horizontal earth pressure at various depths below ground level should be found using a soil–structure interaction analysis. Requirements for such soil-structure interaction analysis methods are given, and it is explained that an approach is included in Annex A that satisfies these requirements. Other approaches satisfying these requirements would be equally acceptable. See also the text relating to Annex A at the end of this paper.

9.4.7 Partial factors

Partial factors in the Eurocodes are applied differently from those in BA 42/96. For K^* pressure, γ_Q is applied to the thermal action as described in the discussion on **9.4.2** above, $K_{p;t}$ is factored by the reciprocal of γ_M (γ_M *), (see discussion on **9.4.1** above) and the resulting pressure is effectively factored by γ_G for the weight of soil. The model factor γ_{SdK} is not applied to K^* pressure (see Denton et al[3]). Other partial factors are combined as described for Design Approach 1 in BS EN 1997-1 **2.4.7.3.4.2.**

9.4.8 Pressure envelope

For many elements of integral bridges, minimum pressures are more critical than maximum pressures, and contraction distortions are more critical than expansion distortions. It is therefore necessary to consider a range of combinations of pressures and thermal movements. Figure 6 in the PD illustrates the range of pressures and movements that need to be considered.

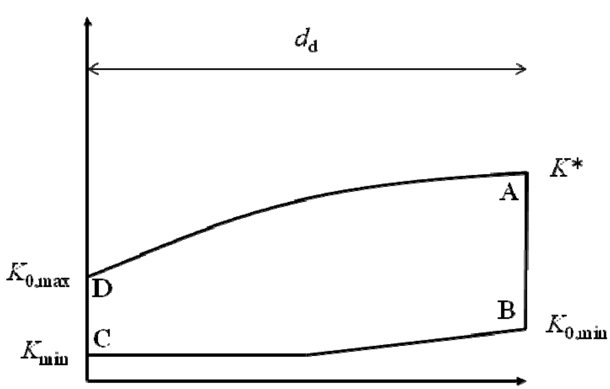


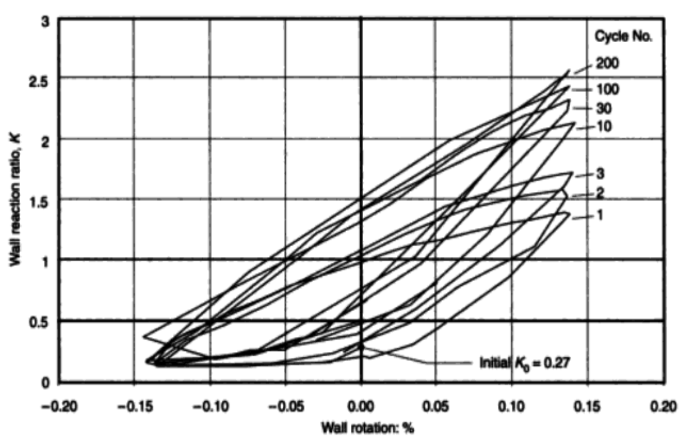
Figure 4. Pressure envelope (after Figure 6 of PD 6694-1)

Point A is the position of maximum expansion and maximum (K^*) pressure.

When expansion stops and contraction starts, the direction of friction in the backfill reverses and the pressures reduce rapidly. It is assumed in the PD that early in the life of the bridge a lower bound value of at rest pressure may occur in conjunction with maximum expansion (Point B). This is conservatively low compared with the plots in Figure 5 below, but it takes account of possible vertical soil arching on the back of the abutment.

As contraction proceeds the pressure on the back of the wall will further reduce until it reaches minimum active pressure (K_{min}). This value is maintained until the maximum contraction position (Point C) is reached.

When contraction stops and expansion starts, friction in the backfill again reverses and active pressure rapidly increases to at rest pressure. Figure 6 in the PD shows an upper bound value of at rest pressure being applied in conjunction with maximum contraction (Point D). Recent work has shown that this may be somewhat conservative because a small but significant expansion movement is required to mobilise at rest pressure when the direction of movement changes.



Pressure Cycles after England et al[6]: Variation in wall reaction ratio K during cyclic wall rotations of amplitude 0.125% for 200 cycles

Figure 5. Experimental pressure/rotation plots

9.5 Longitudinal loads

9.5.1 Traffic surcharge with integral bridges

Traffic surcharge is not required to be considered with K^* pressure. This has been included partly for pragmatic reasons considering past design practice, but also has some physical justification that may be understood by considering the direction of frictional effects acting within the soil behind integral bridge abutments. The expansion of the bridge deck necessary to develop K^* pressures give rise to frictional effects in the opposite direction from those that tend to arise from traffic surcharge (unless the soil reaches a fully passive state). As a result traffic surcharge effects and K^* pressures are not wholly additive.

9.5.2 Resistance to longitudinal load.

The values of the wall rotations required to mobilise full passive and half passive pressure given in Annex C to BS EN 1997-1 are very large compared to those given by Hambly (as discussed by Denton et al[3]). Caution should therefore be exercised where the safety of the structure is reliant on any specific degree of passive pressure.

9.7.2 Spread footings for full height frame abutments

The effects of the sliding of the base slab of a full height abutment needs to be carefully considered because, although the K^* pressures may reduce if the base slab slides forward, if the base slab does not slide back, the K^* pressures will subsequently build up again with time. In this case the effects of both the K^* pressure and the distortions due to the sliding displacement have to be considered. Alternatively, if the base slab could slide in one direction and then slide back again, the K^* pressure associated with end screen abutments may be more appropriate.

9.7.3 Spread footings for bank pad abutments

The PD requires a reduction in bearing resistance for foundations subject to sliding.
A similar requirement is in BA 42/96.

9.7.4 Piled Foundations

The restriction in BA 42/96 regarding the use of raked piles appears to be related to the effect on the piles if the abutment has to sway to accommodate thermal movements. The PD only allows raking piles to be used in a configuration that will preclude sway.

9.8 Skew Effects

9.8.1 Twisting of a frame abutments and integral piers

Figure 7 in the PD relates to wall abutments and leaf piers which are integral or rigidly connected to the deck. In most realistic cases these are capable of accommodating the angle of twist, ζ, but it is important to check the deck/substructure connection, and there is record of bearings which have failed as a result of this effect.

For piers or abutments consisting of columns which are flexible about both axes this effect does not occur because these types of support are free to flex in the direction of expansion.

9.8.2 Plan rotation due to earth pressure on the ends of skew bridges

Figure 6a shows that the twisting moment caused by the non-aligned earth pressure action P on the opposite abutments of a skew integral bridge is $PL\sin\theta$ and that a force, such as $R = P\tan\theta$ perpendicular to P, would be necessary to resist this couple.

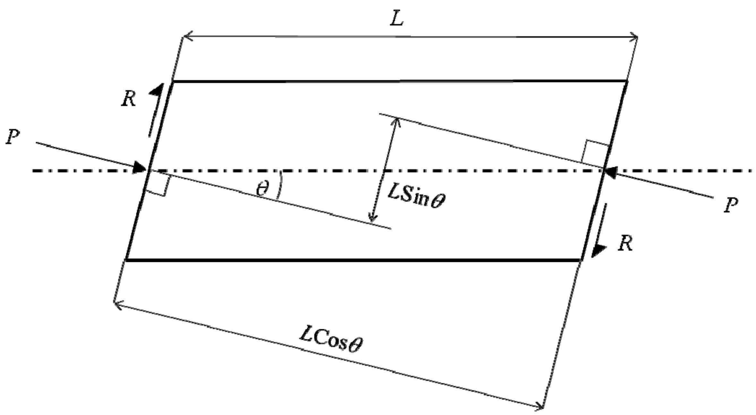


Figure 6a. Twisting on skew bridges

This effect is particularly important in regard to skewed bank seat abutments. If for a skewed bank pad abutment *R* is assumed to be provided only by the frictional resistance of the base, then, when the bridge starts to expand and slide over the foundation, the whole friction force will immediately align itself with the direction of movement of the ends of the deck. If the deck were to expand without twisting, *R* would then act parallel to the direction of expansion (*i.e.* along the centre line of the deck) and there would be no couple to resist the torque applied by the non-aligned earth pressures. What would happen in practice is that the bridge would twist as it expanded, as shown in Figure 6b, points A and B would move towards points A' and B' respectively, the friction forces would act along A'A and B'B providing a couple $RL\sin\lambda$ where λ is the angle of twist. The value of λ would be such that $RL\cos\lambda$ equalled $PL\sin\theta$.

Figure 6c shows the same situation for contraction with point A moving along AA" and point B moving along BB" so that the bridge twists in the same direction as for expansion. From this is can be seen that a skew bank pad abutment which relies on base friction to resist the twisting effects due to earth pressure, will twist through an angle λ every time the bridge deck expands or contracts and these twisting movement will be accumulative. For this reason friction on the underside of a bank pad abutment must be ignored when providing resistance to plan rotation or other lateral movement.

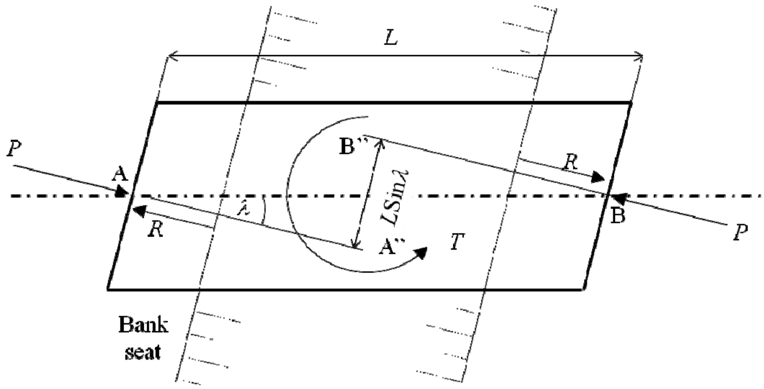


Figure 6b. Expansion case

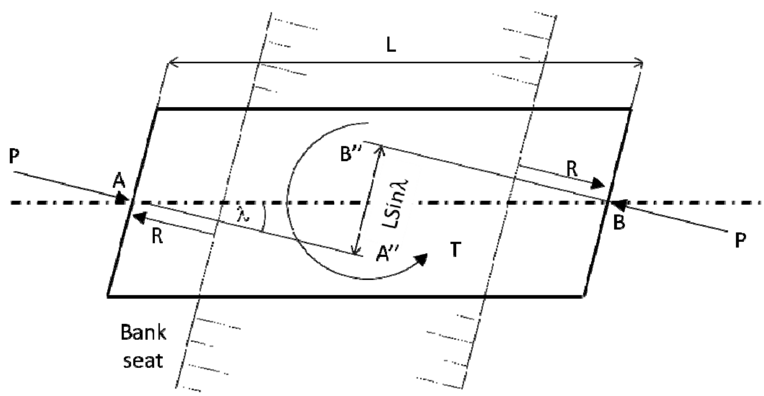


Figure 6c. Contraction case

Wing walls (9.9)

Wing Walls supporting *K** pressures (9.9.2)

The ratio of effective soil pressure between two orthogonal frictionless faces (*i.e.* between effective principal stresses) is $1/K_a$ where K_a is the Rankine value of the active pressure coefficient. If K_{ww} is the pressure coefficient on a wing wall orthogonal to an end screen and K^* is the pressure coefficient on the end screen then $K^*/K_{ww} = 1/K_a$ and therefore $K_{ww} = K_aK^*$ as in **9.9.2** of the PD.

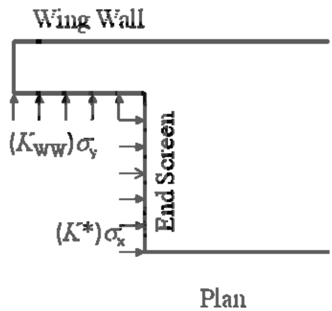


Figure 7. Wing walls support *K pressures**

Wing Walls at acute corners

The second paragraph of **9.9.2** says that where the wing wall encloses an acute corner as shown in **Figure 8** of the PD, the equilibrium of the earth wedge ABC should be considered when determining the wing wall pressures. It is axiomatic that when a soil (or liquid) is retained by vertical frictionless walls or surfaces, the pressure on all walls is the same. If therefore in Figure 8, K^* pressure develops on AB then K^* pressure will also develop on AB and BC. Wall friction, and soil-to-soil friction on AB will reduce these pressures, but it is likely that for acute corners, the earth pressure coefficients on the wing wall will be greater than value of K_aK^* given in **9.9.2**.

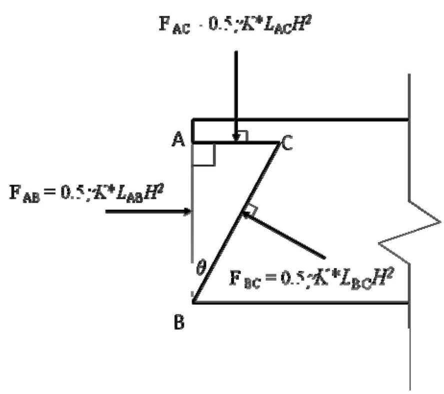


Figure 8. Equilibrium of soil wedge in acute corner between wing wall and abutment

Rock cuttings (9.10.3)

Although the slip plane of a passive soil wedge occurs at an inclination much less than 45^o it has been shown that for integral bridges, the re-orientation of particles which leads to the build up K^* pressure all occur within a 45^o wedge.

Designing integral abutments using a soil–structure interaction analysis (Annex A)

Annex A gives a method for designing integral bridges using a soil-structure interaction analysis. This method is based on proposals developed by Arup[13] and satisfies the criteria given in **9.4.5**. As explained in the discussion of **9.4.5** above, other methods that fulfil these criteria would be equally acceptable. An illustration of the output from one stage in the analysis method is shown in Figure 9.

Defining soil parameters

The properties for the soils at the back and the front of the abutment over depths H' are derived from the corresponding soil 'strain' d_d'/H' (as defined in **Figure A.2**), where d_d' is the wall deflections at mid depth ($H'/2$) when the end of the deck moves through a distance d_d, and H' is the depth of backfill affected by the thermal cycling. H' is different for the front and back of the abutment. For an abutment on a spread footings $H' = H$ as shown in **Figure 5** of the PD. For piles or embedded foundations, H' is taken as the depth from surface level to the level at which the soil pressure reduces to at rest pressure.

The PD provides guidance on calculating the variation in soil stiffness with depth for a given applied soil strain, account for:

- the increase in the stiffness of granular soils with each repeated cycle, based on Clayton et al[9]; and that,
- soil stiffness is a function of both the soil strain, and the mean effective stress of the soil (*i.e.* soil stiffness increases with depth), based on Seed and Idris[17]

The variation in soil stiffness with depth may be modelled in an FE analysis packages by splitting the soil into a series of discrete horizontal layers, and assigning the appropriate value of Young's Modulus (E_G) in accordance with **A.3.2**. The effect of strain ratcheting is taken into account by multiplying E_G by 1.5 (to give $E_{G,d}$). The soil properties for each layer should also include the quasi-passive limit K^*_d derived from the appropriate soil 'strain' parameters d_d'/H' in accordance with **A.3.3**.

Determining d_d'/H'

For abutment forms such as flexible wall abutments and abutments founded on a single row of piles where the soil 'strain' parameter d_d'/H' cannot be readily identified from the abutment geometry, an iterative procedure is provided in **A.4.3** to ensure compatibility between the soil strains and soil properties. The procedure provides initial assumptions regarding H' and d_d' and involves an initial contraction of $0.5d_d$. It is essential that this initial contraction is applied to achieve realistic results.

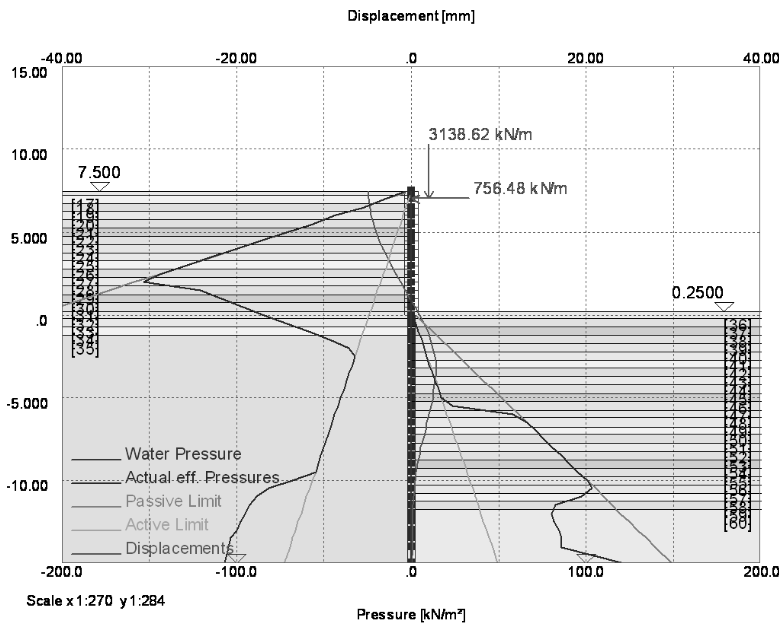


Figure 9: Soil structure interaction analysis of abutment moving into the backfill

Conclusions

The Eurocodes do not cover important aspects of the design of integral bridges. PD 6694-1 therefore contains recommendations and guidance for the design of integral bridges. The background to these recommendations has been given, with comparisons made to those in BA 42/96.

Acknowledgements

The authors wish to acknowledge the contributions of their colleagues at Parsons Brinckerhoff and Arup to the background studies and the development of the PD.

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DESIGN ILLUSTRATION – BRIDGE ABUTMENT DESIGN

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Abstract
This paper provides a calculation showing how the heel length and overall length of the base slab of a conventional cantilever gravity abutment can be determined in accordance with the requirements of the Eurocodes and relevant non-contradictory information[1],[2],[3],[4],[5],[6]. Sliding resistance, bearing resistance and overturning stability are all considered.
The calculations illustrate the requirements of the Eurocodes in regard to loading, partial factors, combination of actions and other issues which require a somewhat different approach from that used with pre-Eurocode designs.

Notation
The symbols used in the calculations are as for the Eurocode and PD 6694-1. Other symbols are defined in the text of the calculations or identified in the Figure 1.

The Design Problem
An 8m high, 12m wide abutment of a multispan continuous bridge is shown in Figure 1. It is required to determine the heel length (B_{heel}) and the overall base length (B) of this abutment necessary to satisfy the requirements of sliding resistance, bearing resistance and overturning stability specified in the Eurocodes. The abutment is subject to three notional 3m wide lanes of traffic surcharge.

The characteristic actions and soil parameters applied to the abutment are as follows:

Permanent actions

Weight of steel beams		50	kN/m
Weight of concrete deck		72	kN/m
Weight of surfacing		36	kN/m

Actions for traffic group gr2

Maximum vertical traffic reaction	$V_{traffic}$	100	kN/m
Uplift due to traffic on adjacent span	$U_{traffic}$	30	kN/m
Braking and acceleration action	$H_{braking}$	50	kN/m
UDL surcharge (from PD 6694-1 Table 5)	σ_h	20K$_a$	kN/m²
Line load surcharge (from PD 6694-1 Table 5)	F	2 x 330 K$_a$	kN/lane

Soil parameters:

Granular backfill

Weight density	γ_{bf}	18	kN/m²
Angle of shearing resistance	ϕ'_{bf}	35°	

Clay foundations

Weight density	γ	18	kN/m²
Undrained shear strength	c_u	100	kN/m²
Angle of shearing resistance	ϕ'	27°	
Critical state angle of shearing resistance	ϕ'_{cv}	23°	
Overburden pressure (q) = γ_{bf} x Z_q	q	12	kN/m²

The initial dimensions of the foundations are to be based on traffic load group gr2 in which the characteristic value of the multi-component action is taken as the frequent value of Load Model 1 in combination with the frequent value of the associated surcharge model, together with the characteristic value of the braking and acceleration action, (see the UK National Annex to BS EN 1991-2:2003, **NA.2.34.2**).

Wind is not required to be considered in combination with traffic model gr2 and thermal actions are not considered to be significant and are therefore neglected in these preliminary calculations.

The water table is well below foundation level and need not be considered, but it is required to check sliding resistance and bearing resistance at STR/GEO for both the drained and the undrained condition.

As no explicit settlement calculation is to be carried out at SLS it is required to be demonstrated that a sufficiently low fraction of the ground strength is mobilised (see BS EN 1997-1:2004, **2.4.8(4)**). This requirement will be deemed to be satisfied if the maximum pressure at SLS does not exceed one third of the characteristic resistance (see PD 6694-1, **5.2.2**).

Transverse Dimensions

Abutment width W_{abut} = 12m

Notional lane widths W_{lane} = 3m

The traffic on the third notional lane is subject to a 0.5 lane factor so that the effective number of lanes N_{lane} used in the surcharge calculation is 2.5 (see UK National Annex to BS EN 1991-2:2003, **NA 2.34.2**)

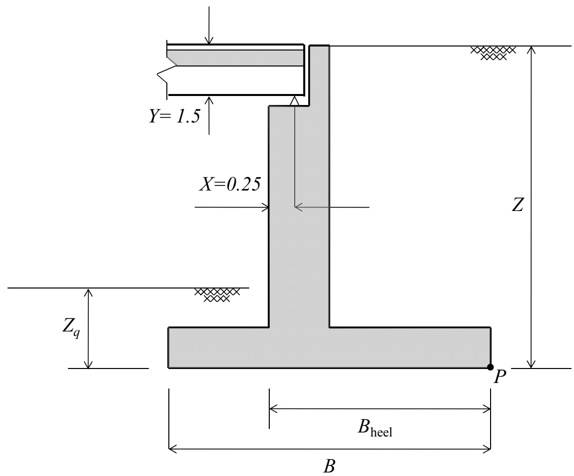

Figure 1. Base slab design for a gravity cantilever bridge abutment

Methodology for Preliminary Design

Calculations are carried out "in parallel" for the SLS characteristic combination of actions and for STR/GEO Combinations 1 and 2, using Design Approach 1 (see BS EN 1997-1:2004, **2.4.7.3.4.2**). Partial factors on actions are taken from the UK National Annex to BS EN 1990:2002, **Table NA.A2.4 (B) and (C)**, partial factors for soil parameters are from the UK National Annex to BS EN 1997-1:2004, **Table A.NA.A.4** and ψ factors from the UK National Annex to BS EN 1990:2002, **Table NA.A2.1**. The preliminary calculations are carried out on a "metre strip" basis

The following procedure was used for the preliminary design:

1. Calculations were carried out "in parallel" for the SLS characteristic combination and for STR/GEO Combinations 1 and 2. This allowed a side-by-side comparison of the three limit states to be made and repetitive calculations to be minimised.

2. The following actions were calculated:

(a) The total horizontal action on the wall (H) due to active earth pressure, traffic surcharge (factored by ψ_1) and braking and acceleration (see Table 1).

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(b) The minimum vertical reaction due to deck reaction ($V_{DL;inf}$) and uplift caused by traffic on remote spans (U) (see Table 2).

(c) The maximum vertical reaction due to the weight of the deck and traffic (see Table 3).

(d) The vertical pressure exerted by the backfill and the base slab ($\gamma_{bf}Z$). (For convenience in these preliminary calculations, the density of the concrete in the base slab and abutment wall was considered to be the same as the density of the backfill (γ_{bf})).

3. The length of the heel (B_{heel}) required to provide enough weight to resist sliding for the drained foundation was found as follows:

The sliding resistance due to the weight of the deck less traffic uplift (R_{vx}) was taken as $(V_{DL;inf} - U)tan\phi'_{cv}$. The required sliding resistance due to the weight of the backfill and abutment was therefore H-R_{vx}. The weight of the abutment and backfill required to provide this resistance was therefore equal to $(H\text{-}R_{vx})/tan\phi'_{cv}$ and this had to equal $B_{heel}\gamma_{bf}Z$. The required value of B_{heel} therefore equalled $(H\text{-}R_{vx})/(\gamma_{bf}Ztan\phi'_{cv})$ and this equals $(H\text{-}R_{vx})/(\mu\gamma_{bf}Z)$ as in Table 2.

4. As it was recognised that the loads on the toe and the use of the correct density of concrete would increase the sliding resistance, the selected figure of B_{heel} in Table 2 was taken as slightly less than the figure of B_{heel} obtained from the calculation.

5. For undrained foundations the total overall base length for sliding, ($B1$) was taken as $H_d/c_{u;d}$ as in Table 2 (see BS EN 1997-1:2004, **Equation 6a**).

6. The required *overall* base length is also dependent on other factors such as the requirement to keep the load within the middle third at SLS and within the middle two-thirds at ULS, the bearing resistance for the drained and undrained condition and in some circumstances (although not for this structure) for resistance to overturning. In all these calculations the eccentricity of the vertical action is required. To obtain this it is convenient to take moments about the back of the heel (point P on Figure 1) rather than the centre of the base, because the bearing resistance calculations are iterative, but the moments about the back of the heel do not alter with varying toe lengths, provided the value of B_{heel} is not changed. This allows multiple iterations to be carried out with minimal change to the data.

7. Moments about P were calculated (see Tables 4 and 5). The distance of the line of action from P is e_{heel}, where $e_{heel} = M/V$ and M is the total moment about P and V is the total vertical load. It can be shown that to satisfy the SLS middle third condition (see PD 6694-1, **5.2.2**), the overall base length ($B2$) must be 1.5 e_{heel}, and to satisfy the ULS middle two-thirds condition (see BS EN 1997-1:2004, **6.5.4**), the overall base length ($B3$) must be 1.2 e_{heel} (see Table 5).

8. To determine the overall base length (B) required to provide adequate bearing resistance for the undrained and drained conditions, an iterative calculation with increasing values of B was carried out, starting with the maximum value of the base length found from the sliding calculations (i.e the largest of $B1$, $B2$, or $B3$) and increasing progressively until the bearing resistance for the undrained and drained conditions drained and the toe pressure limitation at SLS were all satisfied.

9. The selected value of B and the calculations in Table 6 and 7 are based on the final iteration, that is the minimum value of B necessary to satisfy the bearing resistance requirements. The calculations largely replicate the equations given in BS EN 1997-1:2004, **Annex D**.

Notes

- The abutment is assumed to be transversely stiff and so the traffic loads can be distributed over the whole width of the abutment (see PD 6694-1, **Table 5 Note C**)
- For convenience in the preliminary design, the density of the concrete in the base slab and wall is considered to be the same as the density of the backfill (γ_{bf}).
- It should be noted that the same partial factor γ_G is applied to the vertical and horizontal earth pressure actions (see PD 6694-1, **4.6**). This is only likely to be relevant in a sliding resistance calculation if STR/GEO Combination 1 is more critical than Combination 2.
- In these calculations a model factor, $\gamma_{Sd;k}$ = 1.2 has been applied to the horizontal earth pressure at ULS in order to maintain a similar level of reliability to previous practice (see PD 6694-1, **4.7**).
- In Tables 1 to 7 the figures given in the SLS column are the characteristic values of material properties and dimensions and the characteristic or representative values of actions per metre width. The figures in the STR/GEO columns are the design values unless otherwise indicated.

Horizontal actions		**SLS**	**STR/GEO**		
			Comb. 1	**Comb. 2**	
Height of abutment Z	Z	8.00	8.00	8.00	m
Partial factor on soil weight	$\gamma_{G;sup}$	1.00	1.35	1.00	
Backfill density = $\gamma_{bf;k}\,\gamma_{G;sup}$	$\gamma_{bf,d}$	18.0	24.3	18.0	kN/m^3
$\phi'_{bf;k}$ = 35°; Partial factor γ_M on $\tan(\phi'_{bf;k})$	γ_M	1.00	1.00	1.25	
$\tan^{-1}(\tan(\phi'_{bf;k})/\gamma_M) = \phi'_{bf;d}$	$\phi'_{bf;d}$	35.0	35.0	29.3	
Active pressure coefficient $\boldsymbol{K_a}$ incl. (γ_M) $(1-\sin\phi'_{bf;d})/(1+\sin\phi'_{bf;d})$	$\boldsymbol{K_a}$	0.27	0.27	0.34	
Model factor $\gamma_{Sd:K}$	$\gamma_{Sd:K}$	1.00	1.20	1.20	
Design active pressure action $\gamma_{bf;d}K_a\gamma_{Sd;K}Z^2/2 = H_{ap;d}$	$\boldsymbol{H_{ap;d}}$	156	253	237	kN/m
Surcharge UDL = $\sigma_h W_{lane} N_{lane}/W_{abut}$ = $(20K_a)$ x 3 x 2.5/12 = $\sigma_{h;ave}$	$\sigma_{h;ave}$	3.39	3.39	4.29	kN/m^2
Surcharge UDL action $\sigma_{h;ave}$ x Z = $H_{sc;udl}$	$H_{sc;udl}$	27.1	27.1	34.3	kN/m
Surcharge Line Load/m = $F\,K_a N_{lane}/W_{abut}$ = 2 x $330K_a$ x 2.5/12 = $H_{sc;F}$	$H_{sc;F}$	37.3	37.3	47.2	kN/m
Combined surcharge/m $H_{sc;udl} + H_{sc;F}$ = $H_{sc;comb}$	$H_{sc;comb}$	64.4	64.4	81.6	kN/m
Partial factor on surcharge γ_Q	γ_Q	1.00	1.35	1.15	
ψ_1 = 0.75 for surcharge in traffic group grp2	ψ_1	0.75	0.75	0.75	
Design surcharge = $H_{sc;d} = H_{sc;comb}.\psi.\gamma_Q$	$\boldsymbol{H_{sc;d}}$	48.3	65.2	70.4	kN/m
Characteristic braking action $H_{braking;k}$	$H_{braking;k}$	50.0	50.0	50.0	kN/m
Partial factor on braking γ_Q	γ_Q	1.00	1.35	1.15	
Braking action /m $H_{braking;d} = H_{braking;k}\,\gamma_Q$	$\boldsymbol{H_{braking;d}}$	50.0	67.5	57.5	kN/m
TOTAL DESIGN HORIZONTAL ACTION $H_d = H_{ap;d} + H_{sc;d} + H_{braking;d}$	$\boldsymbol{H_d}$	254	386	365	kN/m

Table 1. Horizontal actions

Minimum vertical actions and sliding resistance		**SLS**	**STR/GEO Comb. 1**	**STR/GEO Comb. 2**	
Height of abutment Z	Z	8.00	8.00	8.00	m
Characteristic deck weight 50+72+36 = 158	$V_{DL;k}$	158	158	158	kN/m
Inferior partial factor on deck weight	$\gamma_{G;inf}$	1.00	0.95	1.00	
Inferior weight of deck	$V_{DL;inf;d}$	158	150	158	kN/m
Uplift from traffic	U_k	30.0	30.0	30.0	kN/m
Superior partial factor on uplift	$\gamma_{Q;sup}$	1.00	1.35	1.15	
ψ_1= 0.75 for vertical traffic actions in traffic group grp2	ψ_1	0.75	0.75	0.75	
Factored uplift from traffic $U_{k.}\gamma_Q\ \psi = U_d$	U_d	22.5	30.4	25.9	kN/m
Minimum vertical loads from deck and traffic $V_{x;d} = V_{\mathrm{DL;inf;d}} - U_d$	$\boldsymbol{V_{x;d}}$	136	120	132	kN/m
$\phi'_{cv;k}$ = 23°; Partial factor γ_M on $\tan(\phi'_{cv;k})$	γ_M	1.00	1.00	1.25	
Coefficient of friction $\tan(\phi'_{cv;k})/\gamma_M = \mu_d$	μ_d	0.42	0.42	0.34	
Sliding resistance due to $\mathbf{V_x}$ $\mu_d.V_{x;d} = R_{vx;d}$	$R_{vx;d}$	57.5	50.8	44.9	kN/m
Horizontal action from Table 1, H_d	H_d	254	386	365	kN/m
Reqd resistance from backfill $[H_d - R_{vx;d}]$	R_{req}	197	335	320	kN/m
Density of backfill (Table 1) $\gamma_{bf;d}$	$\gamma_{b;df}$	18.0	24.3	18.0	kN/m^3
Frictional shear stress due to backfill: $[\mu_d\gamma_{bf;d}\ Z]$		61.1	82.5	48.9	kN/m^2
Required $B_{heel} = R_{req} / (\mu_d\ \gamma_{bf;d}\ Z)$	$B_{heel;req}$	3.22	4.06	**6.55**	m
SELECTED VALUE OF HEEL $\boldsymbol{B_{heel}}$ Rounded down, see Methodology para. (4)	$\boldsymbol{B_{heel}}$	**6.25**	**6.25**	**6.25**	m
$c_{u;k}$ = 100; Partial factor γ_M on $c_{u;k}$	γ_{cu}	1	1	1.4	
Undrained shear strength $c_{u:d} = c_{u;k} / \gamma_{cu}$	$c_{u:d}$	100.0	100.0	71.4	kN/m^2
OVERALL BASE LENGTH B1 = $H_d/c_{u;d}$	$\boldsymbol{B1}$	2.54	3.86	**5.11**	**m**

Table 2. Minimum vertical actions and sliding resistance

Maximum vertical actions		SLS	STR/GEO Comb. 1	STR/GEO Comb. 2	
Height of abutment Z	Z	8.00	8.00	8.00	m
Selected value of B_{heel} (Table 2)	B_{heel}	6.25	6.25	6.25	m
Partial factor on steelwork $\gamma_{G;sup}$	$\gamma_{G;sup}$	1.00	1.20	1.00	
Weight of steelwork = $50\gamma_{G;sup}$		50.0	60.0	50.0	kN/m^3
Partial factor on concrete $\gamma_{G;sup}$	$\gamma_{G;sup}$	1.00	1.35	1.00	
Weight of concrete = $72\gamma_{G;sup}$		72.0	97.2	72.0	kN/m^3
Partial factor on surfacing $\gamma_{G;sup}$	$\gamma_{G;sup}$	1.00	1.20	1.00	
Weight of surfacing = $36\gamma_{G;sup}$		36.0	43.2	36.0	kN/m^3
Superior weight of deck/m $V_{DL;sup;d}$	$V_{DL;sup;d}$	158	200	158	kN/m
Characteristic vertical action from traffic $V_{traffic;k}$	$V_{traffic;k}$	100.0	100.0	100.0	kN/m
Partial factor on traffic γ_Q	γ_Q	1.00	1.35	1.15	
$\psi_1 =$ 0.75 for vertical traffic actions in traffic group grp2	ψ_1	0.75	0.75	0.75	
Design traffic action/m $V_{traffic;d} = V_{traffic;k}\,\gamma_Q\,\psi$	$V_{traffic;d}$	75.0	101	86.3	kN/m
Density of backfill $\gamma_{bf;d}$ (Table 1)	$\gamma_{bf;d}$	18.0	24.3	18.0	kN/m^3
Selected width of heel B_{heel} (Table 2)	B_{heel}	6.25	6.25	6.25	m
Design weight of backfill/m $V_{bf;d} = \gamma_{bf;d}\,Z\,B_{heel}$	$V_{bf;d}$	900	1215	900	kN/m
TOTAL MAXIMUM VERTICAL LOAD $V_{max;d} = V_{DL;sup;d} + V_{traffic;d} + V_{bf;d}$	$V_{max;d}$	1133	1517	1144	kN/m

Table 3. Maximum vertical actions

Moments about the underside of the base due to horizontal actions		**SLS**	**STR/GEO Comb. 1**	**STR/GEO Comb. 2**	
Active Pressure action $H_{ap;d}$ including $\gamma_{Sd;K}$ (Table 1)	$H_{ap;d}$	156	253	237	kN/m
Lever arm = $Z/3$		2.67	2.67	2.67	m
Active Moment $M_{ap;d} = H_{ap;d}$ Z/3	$\boldsymbol{M_{ap;d}}$	416	674	633	kNm
Surcharge UDL action $H_{sc;udl}$ (Table 1)	$H_{sc;udl}$	27.1	27.1	34.3	kN/m
Lever arm = $Z/2$		4.00	4.00	4.00	m
$H_{sc;udl}$ x $Z/2 = M_{sc;udl}$	$M_{sc;udl}$	108	108	137	kNm
Surcharge Line Load $H_{sc;F}$ (Table 1)	$H_{sc;F}$	37.3	37.3	47.2	kN/m
Lever arm = Z	Z	8.00	8.00	8.00	m
$H_{sc;F}\, Z = M_{sc;F}$	$M_{sc;F}$	298	298	378	kNm
Combined surcharge moment $M_{sc;udl} + M_{sc;F}$	$M_{sc;comb}$	406	406	515	kNm
γ_Q for surcharge	γ_Q	1.00	1.35	1.15	
ψ_1 = 0.75 for surcharge in traffic group grp2	ψ_1	0.75	0.75	0.75	
Design surcharge moment $M_{sc;comb}\gamma_Q\psi_1$	$\boldsymbol{M_{sc;d}}$	305	412	444	kNm
Braking action/m $H_{braking;d}$ (Table 1)	$H_{braking;d}$	50.0	67.5	57.5	kN/m
Lever arm for braking $(Z\text{-}Y) = L_{a;b}$ = 8 - 1.5	$L_{a;b}$	6.50	6.50	6.50	m
Braking moment $M_{braking;d}$ $= H_{braking;d}$ x $L_{a;b}$	$\boldsymbol{M_{braking;d}}$	325	439	374	kNm
MOMENT DUE TO HORIZONTAL ACTIONS, $\boldsymbol{M_{hor;d}} = M_{ap;d} + M_{sc;d} + M_{braking;d}$	$\boldsymbol{M_{hor;d}}$	1046	1525	1451	kNm

Table 4. Moments about the underside of the base due to horizontal actions

Moments about the back of the heel		**SLS**	**STR/GEO Comb. 1**	**STR/GEO Comb. 2**	
Width of heel B_{heel}	B_{heel}	6.25	6.25	6.25	m
Distance of deck reactions behind front of wall X	X	0.25	0.25	0.25	m
$L_{a;deck} = B_{heel} - X$	$L_{a;deck}$	6.00	6.00	6.00	m
Superior weight of deck $V_{DL;sup;d}$ (Table 3)	$V_{DL;sup;d}$	158	200	158	kN/m
Deck Moment $V_{DL;sup;d}\, L_{a;deck} = M_{deck;d}$	$M_{deck;d}$	948	1202	948	kNm
Traffic Load $V_{traffic;d}$ (Table 3)	$V_{traffic;d}$	75.0	101	86.3	kN/m
Traffic Moment $V_{traffic;d}\, L_{a;deck} = M_{traffic;d}$	$M_{traffic;d}$	450	608	518	kN/m
Weight of backfill $V_{bf;d}$ (Table 2)	$V_{bf;d}$	900	1215	900	kNm
Backfill moment $V_{bf;d}\, B_{\mathrm{heel}}/2 = M_{bf;d}$	$M_{bf;d}$	2813	3797	2813	kN/m
Total Moment about heel due to vertical actions $\boldsymbol{M_{vert;d}} = M_{deck;d} + M_{traffic;d} + M_{bf;d}$	$\boldsymbol{M_{vert;d}}$	4211	5607	4278	kNm
Moment about base due to horizontal Actions (Table 4)	$\boldsymbol{M_{hor;d}}$	1046	1525	1451	kNm
Total design moment about heel $\boldsymbol{M_{vert;d} + M_{hor;d} = M_{heel;d}}$	$\boldsymbol{M_{heel;d}}$	5257	7131	5729	kNm
Total vertical load V_d (Table 3)	V_d	1133	1517	1144	kN/m
Line of action in front of heel $e_{heel} = M_{heel} / V$	e_{heel}	4.64	4.70	5.01	m
Total length $B2$ required for middle third at SLS = 1.5 e_{heel} (see PD 6694-1 **5.2.2**)	$B2$	**6.96**			m
Total length $B3$ for middle two thirds at ULS = 1.2 e_{heel} (see BS EN 1997-1 **6.5.4**)	$B3$		5.64	**6.01**	m

Table 5. Moments about the back of the heel (Position P on Figure 1)

Bearing Resistance – undrained foundation		SLS	STR/GEO Comb. 1	STR/GEO Comb. 2	
Geometry of foundation (m)					
Final B *(found iteratively)*	B	**8.60**	**8.60**	**8.60**	m
Heel length B_{heel} (Table 2)	B_{heel}	**6.25**	**6.25**	**6.25**	m
Transverse width of foundation L	L	12.0	12.0	12.0	m
Inclination α	α	0^o	0^o	0^o	
Partial factors					
γ_F applied to γ	γ_G	1.00	0.95	1.00	
γ_M applied to $tan\phi'$	$\gamma_{\phi'}$	1.00	1.00	1.25	
γ_M applied to c_u	γ_{cu}	1.00	1.00	1.40	
γ_M applied to c'	$\gamma_{c'}$	1.00	1.00	1.00	
Properties of foundation material					
Weight density γ_d (including γ_G)	γ_d	18.0	17.1	18.0	kN/m^3
Angle of shearing resistance ϕ'_d	ϕ'_d	27.0	27.0	22.2	
Cohesion intercept c'_d	c'_d	0	0	0	
Undrained shear strength $c_{u;d}$	$c_{u;d}$	100.0	100.0	71.4	kN/m^2
Applied action					
Horizontal actions H_d (Table 1)	H_d	254	386	365	kN/m
Vertical action V_d (Table 3)	V_d	1133	1517	1144	kN/m
Moment about $P = M_{heel;d}$ (Table 4)	$M_{heel;d}$	5257	7131	5729	kNm/m
$M_{heel;d}/V = e_{heel}$	e_{heel}	4.64	4.70	5.01	m
Eccentricity about centre line $e = e_{heel} - B/2$	e	0.34	0.40	0.71	m
Overburden pressure	q_d	12.0	12.0	12.0	kN/m^2
Effective foundation dimensions (m)					
Effective foundation breadth $B' = B$-2e	B'	7.92	7.80	7.19	m
Effective area for 1m strip design $A' = B'$	A'	7.92	7.80	7.19	m^2
Effective transverse width $L' = L$	L'	12.0	12.0	12.0	m
Undrained bearing resistance (Annex D to BS EN 1997-1 **D.3**)					
Bearing parameters for undrained foundations					
$b_c = 1-2\alpha/(\pi+2)$	b_c	1.00	1.00	1.00	
$s_c = 1+0.2(B'/L')$	s_c	1.13	1.13	1.12	
$i_c = ½\{1+\sqrt{(1-H_d/A'c_{u;d})}\}$	i_c	0.91	0.86	0.77	
$R/A' = (\pi+2)c_{u;d}b_c s_c i_c + q_d$	R/A'	543	509	328	kN/m^2
V_d/A'	V_d/A'	143	195	159	kN/m^2
Ratio $\boldsymbol{R/V}$	$\boldsymbol{R/V_d}$	3.79	2.62	2.06	
Settlement check					
1/3(R/A') at SLS characteristic		181	Not critical (see Table 7)		kN/m^2
Max toe pressure $(1+6e/B)V_d/B$		163			kN/m^2

Table 6. Bearing Resistance – undrained foundation

Bearing Resistance – drained foundation		SLS	STR/GEO		
			Comb. 1	Comb. 2	
Effective foundation dimensions					
B' from Table 6	B'	7.92	7.80	7.19	m
L' from Table 6	L'	12.0	12.0	12.0	m
A' per metre width from Table 6	A'	7.92	7.80	7.19	m^2
Other geometry, partial factors, foundation properties and actions are as Table 6					
Bearing parameters for drained foundations					
$N_q = e^{\pi \tan\phi'd} \tan^2 (45 + \phi'_d/2)$	N_q	13.2	13.2	7.96	
$N_c = (N_q-1)\cot \phi'_d$	N_c	23.9	23.9	17.1	
$N_\gamma = 2 (N_q-1) \tan \phi'$, where $\delta \geq \phi'/2$ (rough base)	N_γ	12.4	12.4	5.68	
$b_c = b_q - (1-b_q)/N_c \tan \phi'_d$	b_c	1.00	1.00	1.00	
$b_q = b_\gamma = (1 - \alpha \tan \phi'_d)$	b_q, b_γ	1.00	1.00	1.00	
$s_q = 1 + (B'/L') \sin \phi'_d$, for a rectangular shape	s_q	1.30	1.29	1.23	
$s_\gamma = 1 - 0.3 (B'/L')$, for a rectangular shape;	s_γ	0.80	0.81	0.82	
$s_c = (s_q N_q - 1)/(N_q - 1)$ for rectangular, square or circular shape	s_c	1.32	1.32	1.26	
$m = (2+B'/L')/(1+B'/L')$	m	1.60	1.61	1.63	
$i_q = [1 - H/(V + A'c'_d \cot \phi'_d)]^m$	i_q	0.67	0.62	0.54	
$i_c = i_q - (1 - i_q)/N_c \tan \phi'_d$	i_c	0.64	0.59	0.47	
$i_\gamma = [1 - H/(V + A'c'_d \cot \phi'_d)]^{m+1}$	i_γ	0.52	0.47	0.36	
R/A′ = (Equation from BS EN 1997-1 D.4)					
$c'_d N_c b_c s_c i_c$		0.00	0.00	0.00	
$q_d N_q b_q s_q i_q$		137	128	62.7	
$0.5 \gamma_d B' N_\gamma b_\gamma s_\gamma i_\gamma$		367	311	110	
R/A' = sum of above	R/A'	504	439	172	kN/m^2
V_d/A'	V_d/A'	143	195	159	kN/m
Ratio R/V	$\boldsymbol{R/V_d}$	3.52	2.25	**1.08**	
Resistance to limit settlement at SLS					
1/3 (R/A') at SLS characteristic		**168**	Limits		kN/m^2
Max toe pressure (1+6e/B) V_d/B		**163**	satisfied		kN/m^2

Table 7. Bearing Resistance – drained foundation

Final Design

After the preliminary design has been completed a final design should be carried out as given below:

1. Select the final dimensions based on the preliminary values: B_{heel} = 6.25m and B = 8.6m. As the weight on the toe has not been included in the preliminary design, B_{heel} has been "rounded down" and the overall length (B) may need to be "rounded up".

2. The final selected base slab dimensions should be verified using the correct concrete densities, the loads on the toe and other relevant combinations of actions.

Details of the final design calculations are not included in this paper.

Conclusions

It is difficult to generalise about which combination of actions or which limit states are critical on the basis of calculations for a single bridge because the critical combination is often determined by the ratio of the bridge span to the abutment height or the ratio of traffic action to the soil actions. It is however clear that horizontal earth pressures are generally critical for STR/GEO Combination 1 regardless of the bridge proportions because γ_G for soil is higher than $K_{a;d}/K_{a;k}$ for most realistic values of ϕ'. Also, for undrained sliding resistance Combination 2 is always likely to be critical because γ_M on c_u is higher than $K_{a;d}/K_{a;k}$ for all realistic values of ϕ', and it is also higher than γ_Q on surcharge braking and acceleration.

For drained sliding resistance, in the calculations presented in this paper, Combination 2 was more critical than Combination 1, primarily because the effects of γ_G on the weight of soil were favourable for sliding resistance and unfavourable for horizontal pressure and therefore, to some extent, cancelled each other out in Combination 1. It was however apparent from the calculations that Combination 1 could be critical for sliding for low abutments supporting long spans where braking and acceleration actions were large and earth pressures were small.

For bearing pressure, Combination 2 was found to be significantly more critical than Combination 1 for both drained and undrained foundations and as γ_M effects tend to predominate in bearing resistance calculations it seems probable that Combination 2 will be critical for bearing resistance in most typical abutments and retaining walls. It was also apparent from supporting calculations that the limitation on toe pressure at SLS is quite severe and that in many cases where it is required to be applied, it will dictate the length of the base. Additional explicit settlement calculations may therefore result in shorter base lengths being required.

Overturning was not found to be an issue for the abutment illustrated in this paper and although it needs to be verified, it appears that it is unlikely to affect the proportions of typical gravity abutments as bearing failure under the toe would normally precede overturning.

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APPLICATION AND FUTURE DEVELOPMENTS

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MAINTENANCE AND FUTURE DEVELOPMENT OF THE EUROCODE

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Abstract

This paper provides an explanation of the processes in place to support the ongoing maintenance of the Eurocodes and outlines plans for their future evolution and promotion outside Europe.

Introduction

With the complete suite of Eurocodes and their UK National Annexes published, the Standards are now subject to ongoing maintenance. It has been agreed by the members of CEN/TC 250 (see Denton[1]) that there should now be a period of substantial stability of the document until 2015. Plans have however been establish for their future evolution and a Mandate for this work has been issued to the European standardisation body, CEN, from the European Commission.

This paper explains the processes in place for the maintenance of the Eurocodes and provides an overview of expected future developments, including the promotion of their use outside Europe.

Maintenance

There are three forms of updates that can be made to CEN Standards, as summarised in Table 1. These are corrigenda, amendment and revision. A similar approach is taken by BSi in the maintenance of British Standards and the UK National Annexes.

Type of Update	Purpose	Denoted by	Voting procedure
Corrigenda	The removal of printing, linguistic or similar errors.	‘Incorporating corrigendum <date>’ added after title.	No enquiry or vote required.
Amendment	Modification, addition or deletion of specific parts of the text.	‘ +A*n*: <date> ’ added after reference number, where *n* is the number of the amendment.	As original document.
Revision	Significant changes to the text.	Standard issued as a new edition with a new date.	As original document.

Table 1: Types of updates to CEN Standards

It is the responsibility of Technical Committees to ensure that all Standards are reviewed periodically, and at intervals not exceeding 5 years. The outcome of such a review can either be to confirm the Standard, issue an amendment, revise the Standards as a new edition with a new date, or to withdraw the Standard.

It was agreed by the members of CEN/TC 250 that there should be a period of substantial stability following March 2010 to allow engineers to gain familiarity with the Eurocodes without this process being hampered by too many updates. As a result there was a significant effort made by the CEN/TC 250 Sub-Committees to publish corrigenda and amendments to address issues that emerged during the development of National Annexes and in the earliest applications before March 2010. In the majority of cases, this target was achieved.

It remain very important, however, for designers to be aware that there are likely to be occasional updates to the Eurocodes and the onus rests with the designer to ensure that they are using the latest versions. The BSi website provides a useful source for this purpose.

It is probably inevitable that corrigenda and amendments to the UK National Annexes will be required to address queries that arise as the use of the Eurocodes becomes more widespread. A balance needs to be struck here between responding to issues in a timely manner and avoiding too many updates. Clearly, if safety issues are found then there will be a heightened urgency to address them.

CEN/TC 250 has agreed that revised Eurocodes will not be published until 2015 at the earliest. However, as discussed below, it seems likely that it will be rather longer before the second generation of EN Eurocodes is ready for publication.

Future developments

Following discussions within CEN/TC 250 and with the European Commission, a medium-term strategy[2] for the evolution of the Eurocodes was developed. Consistent with this strategy, the European Commission issued a Programming Mandate to CEN for the evolution of the Eurocodes[3] in May 2010.

CEN/TC 250 is currently in the process of responding to this Mandate. This is being done by outlining a series of potential projects which, if they are agreed, will form the basis for new specific mandates from the Commission. Considering realistic timescales to finalise the mandates and plans, undertake the necessary work, and complete the CEN enquiry procedures (see Denton[1]), it is unlikely that a second generation of EN Eurocodes will be published before 2018.

The scope of the CEN Programming Mandate is wide ranging. It recognises the potential need both to develop new Eurocodes or Eurocode parts and also to develop further the existing Eurocodes. It responds to needs expressed by European Union Member States to the European Commission through their Eurocode National Correspondents (ENC) (see JRC website for further details[4]).

Specifically, the Mandate includes the:

- extension of existing rules for the assessment of existing buildings and structures and their strengthening;
- design of structures that include structural glass members;
- design of structures that include structural members made of fibre reinforced polymers;
- design of membrane structures;
- extension of existing rules for robustness.

In addition, in connection with the existing Eurocodes, it includes the:

- assessment of all existing Eurocodes concerning the potential to significantly reduce the number of Nationally Determined Parameters (NDPs);
- incorporation of recent results of international studies from scientific and technical associations and results from research programmes relevant to innovation;
- incorporation of recent results of international studies from scientific and technical associations and results from research programmes relevant to contribution of structural design to sustainability;
- adoption, where relevant, of ISO standards to supplement the Eurocode family;
- simplification of rules, where relevant, for limited and well identified fields of application.

The Mandate emphasises the need to respond to feedback from users of the Eurocodes and aid practical local implementation.

In order to develop the project proposal included in the Mandate response on issues relating specifically to bridges, the CEN/TC 250 Horizontal Group – Bridges (see Denton[1]) undertook a consultation exercise with its members and National Contacts, including a workshop in Vienna in October 2010. From this exercise a series of priorities were established that have been included in the draft response to the Mandate.

These include:

- Design of integral bridges
  Integral bridges have become a popular form of construction in many European countries because of their improved durability and the avoidance of expansion joints and some bearings. Key aspects of the design of integral bridges are not currently addressed in the Eurocodes.

- Fatigue verification in bridge design
  EN1991-2 currently includes five fatigue load models, but in practice only one can readily be used in bridge design. There is the possibility for some simplification. Furthermore, based on recent measurements of traffic data, re-calibration of the λ factors used in conjunction with fatigue load model 3 is merited.

- Bridge bearings and expansion joints

Aspects of the design of bridge bearings, in particular the appropriate combinations of actions to be used, are not currently well covered in Eurocodes.

- Robustness requirements in bridge design
  The definition of robustness, relevant actions and requirements given in EN 1990 are not readily usable in bridge design. Based on a review of documented failures and collapses during execution, additional requirements applicable to the design of bridges may be merited to enhance their safety.

- Lateral torsional buckling in bridge design
  Issues have been identified with the treatment of lateral torsional buckling for bridge design that can significantly affect the economy of designs, including the suitability of the buckling curves for plated steel members.

- Partial prestressing and crack control requirements in bridge design
  The suitability of the current provisions for crack control for bridges, particularly prestressed and partially-prestressed structures have been challenged by several HG-Bridges National Contacts. An examination is merited of the need for additional and/or simplified guidance to improve the economy and durability of concrete bridges.

- Footbridge vibrations
  Following issues encountered with several lightweight bridges, notably including the Millennium Bridge in London, a significant amount of new scientific research has been undertaken to develop design approaches to account for pedestrian-induced vibrations. This knowledge is not yet incorporated in the Eurocodes.

- Impact of climate change on environmental actions
  Bridges are long-life assets and their design can be particularly sensitive to environmental actions. Adapting the design of bridges to climate change is likely to offer significant cost and wider sustainability benefits, compared with undertaking later structural modifications.

- Light rail and tram loading models
  Traffic loading models for light rail and tram loading are not currently included in Eurocodes.

- Combination rules for rail / light rail and highway traffic loading
  Combinations rules for bridges carrying rail/light rail and highway traffic are not currently included in Eurocodes.

CEN/TC 250 is in process of organising itself to undertake the work required by the Mandate, including the formation of a series of new working group for the new Eurocodes or Eurocode parts.

Use of Eurocodes outside Europe

There has been increasing interest in recent years in the use of the Eurocodes outside Europe. Many countries outside Europe have previously used the National Standards of European countries rather than develop their own Standards, and with these former National Standards being withdrawn it is natural for such countries to assess their options.

It is clearly in the interest of European countries that the Eurocodes are adopted to replace the withdrawn National Standards. Some countries, such as Singapore, have already committed to the Eurocodes and are in the process of developing their own National Annexes. Other countries have yet to decide, and the European Commission has sought to assist them in this decision making (see Andersson et al[5]).

In November 2009, the European Commission let an 18-month contract (reference ENTR/09/009) to BSi to develop a strategic framework for promoting the adoption and use of Eurocodes in six target regions. The target regions are: Eastern Mediterranean (including the Gulf States); Eastern Europe; South Africa; India; Western Mediterranean; and, South East Asia. At the time of writing this project is ongoing.

Conclusions

With all of the Eurocodes published, the attention of CEN/TC 250 is now focussed on the effective maintenance of the Standards and their future evolution. Ongoing maintenance of the Eurocodes will be undertaken through the publication of corrigenda and amendments, where these are considered absolutely necessary.

Despite the agreement of TC 250 that there should be a period of substantial stability in the Standards, the timescales necessarily for the second generation of EN Eurocodes to be developed dictate that work must commencing now.

The future development of the Eurocodes will be shaped by a Mandate issued to CEN by the European Commission. The Mandate was developed in conjunction with the Commission's Eurocode National Correspondence group, through which the views and priorities of Member States have been expressed.

In addition to promoting the effective use of the Eurocodes in Europe, efforts are also underway by the European Commission to promote the use of the Eurocodes outside Europe.

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