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Background paper to the National Annexes to BS EN 1992-1 and BS EN 1992-3

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Foreword

Publishing information

This Published Document was published by BSI and came into effect on 31 December 2010. It was prepared by Subcommittee B/525/2, *Structural use of concrete*, under the authority of Technical Committee B/525, *Building and civil engineering*. A list of organizations represented on this committee can be obtained on request to its secretary.

Relationship with other publications

This Published Document is a background paper that gives non-contradictory complementary information for use in the UK with the Eurocodes for concrete, BS EN 1992-1-1:2004, BS EN 1992-1-2:2004, BS EN 1992-3 and their UK National Annexes.

NOTE BS EN 1992-1-1 contains general rules applicable to the design of all concrete structures. Therefore, B/525/10, which is responsible for Eurocodes for the design of bridges, was consulted in the drafting of this Published Document.

Presentational conventions

The provisions in this Published Document are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is “should”.

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

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

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

Contractual and legal considerations

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

This Published Document is not to be regarded as a British Standard

Introduction

When there is a need for guidance on a subject that is not covered by the Eurocode, a country can choose to publish documents that contain non-contradictory complementary information that supports the Eurocode. This Published Document provides just such information and has been cited as a reference in the UK National Annexes to BS EN 1992-1-1:2004, BS EN 1992-1-2:2004, and BS EN 1992-3:2006.

PD 6687-2:2008 contains non-contradictory complementary information that supports BS EN 1992-2, parts of which are also relevant to the design of building structures. Designers might find this information useful.

1 Scope

This Published Document is a background paper that gives non-contradictory complementary information for use in the UK with BS EN 1992-1-1:2004, BS EN 1992-1-2:2004, BS EN 1992-3:2006, and their UK National Annexes.

This Published Document gives:

- a) background to the decisions made in the National Annexes for some of the Nationally Determined Parameters;
- b) commentary on some specific subclauses from BS EN 1992-1-1:2004, BS EN 1992-1-2:2004 and BS EN 1992-3:2006; and
- c) reference to the requirements of the building regulations in the UK [1,2] that are not covered in BS EN 1992-1-1:2004, BS EN 1992-1-2:2004, BS EN 1992-3:2006 and their UK National Annexes.

NOTE During the early stages of implementation of BS EN 1992 (all parts), all the related European standards and their UK National Annexes might not be available in their final form. Therefore, guidance on the standards that can be used in conjunction with BS EN 1992-1 (all parts) during the early stages of its implementation is given in Annex A.

2 BS EN 1992-1-1:2004, Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings

2.1 Partial factors for materials [BS EN 1992-1-1:2004, 2.4.2.4]

The values recommended in BS EN 1992-1-1:2004, 2.4.2.4 have been adopted in NA to BS EN 1992-1-1:2004.

BS EN 1992-1-1:2004 gives a value of 1,15 for the partial factor for reinforcement γ_s , which was included in the superseded British Standard for structural use of concrete, BS 8110-1:1997. This value represented a departure from the value of 1,05 that was used in the previous edition, BS 8110-1:1985. The reason for the change revolved around the characteristic strength, to which the partial factor is applied.

For a normal distribution of yield strengths, the characteristic strength f_{yk} is defined as:

$$f_{yk} = f_{ym} - k\sigma_y$$

where

f_{ym} is the mean value of the strength;

σ_y is the standard deviation of the yield strength; and

k is a factor related to the percentage of test results that are likely to fall below f_{yk} .

Traditionally, and in product standards, f_{yk} has been defined as the value below which 5% of the test results are expected to fall. For this percentile the value of k will be 1,64. The statistics from reinforcement production, however, show that the percentage of test results that are likely to fall below a strength of 460 N/mm² [the characteristic value specified in the now withdrawn British Standard for reinforcement BS 4449:1997¹⁾] is much less than 5% and the value of k is commonly nearer 2,5 (see Figure 1). Therefore the mean value of grade 460 reinforcement is approximately $(460 + 2,5\sigma_y)$.

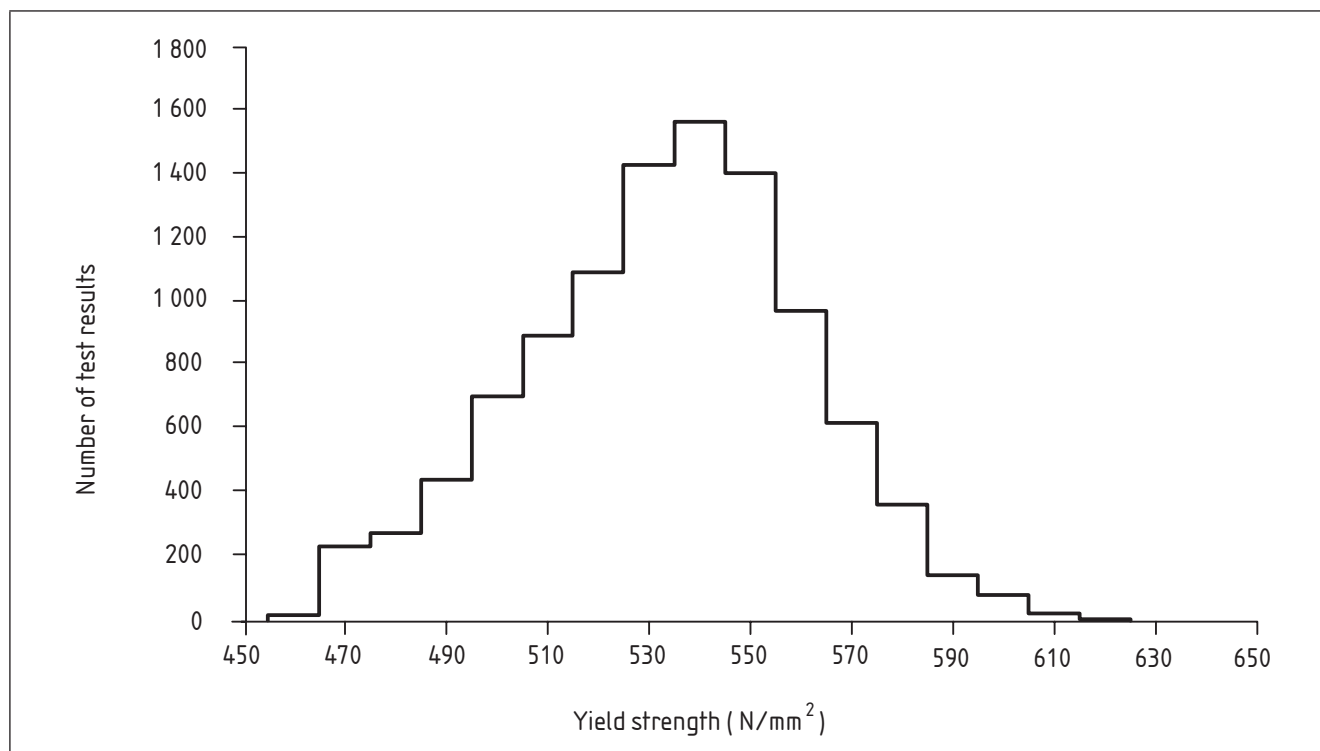
The characteristic value corresponding to the classical 5% value, which is used in the European reinforcement standard BS EN 10080:2005, is much nearer 500 N/mm². The current British Standard for reinforcement, BS 4449:2005, which operates in conjunction with BS EN 10080:2005, now also specifies a characteristic strength of 500 N/mm². Until statistical data for production specified as having this higher characteristic value becomes available, the partial factor of 1,15 should be applied. Therefore there is very little material change to the design values because $500/1,15$ is almost the same as $460/1,05$.

In summary, there is no change in the material property or in the level of safety. The higher value of the partial factor merely represents the value appropriate to the characteristic value of the yield strength, which conforms to the definition in BS EN 10080:2005.

NOTE One of the reasons for the high level of safety in the design stress for reinforcement was the difference in the relationship between the characteristic strength of reinforcement relative to the actual distribution of yield strengths compared with that specified in reinforcement standards such as BS 4449. This needs to be reanalysed when there is more experience of production to new standards. A brief summary of the procedure adopted for fixing the partial factor γ_s is as follows.

- a) The value of 1,05 specified in BS 8110-1:1985 arose from an extensive study carried out for the Department of the Environment in the early 1990s. The mean value and the coefficient of variation for resistance were calculated for some 3 000 reinforcement bar sections using data from surveys of variations in section dimensions, variations in material properties and the model uncertainty in prediction equations.
- b) The global target reliability index β of 3,8 was split into component parts for load effects $\alpha_E\beta$ and resistances $\alpha_R\beta$ as recommended in BS EN 1990:2002+A1:2005. This gives 3,04 as the appropriate target reliability index for the resistance side of the safety equation $\alpha_R\beta$. It was found that a value of 1,15 for γ_s gave a value in excess of 4 for $\alpha_R\beta$ rather than the generally accepted value of 3,04. It was therefore decided that a reasonable value for γ_s was 1,05, which corresponded to a value of 3,8 for $\alpha_R\beta$, still in excess of the target value of 3,04.

¹⁾ BS 4449:1997 has been withdrawn and superseded by BS 4449:2005

Figure 1 Distribution of yield strengths of reinforcement²⁾

2.2 Elastic deformation properties of concrete [BS EN 1992-1-1:2004, 3.1.3]

BS EN 1992-1-1:2004, 3.1.3 recognizes that elastic deformations of concrete largely depend on its composition (particularly aggregates). Therefore the values given in the BS EN 1992-1-1:2004 for creep, shrinkage and elastic modulus should be treated as indicative. These should prove satisfactory for the majority of normal structures. However, it is intended that testing should be undertaken to ascertain the properties of concrete composition used in structures that are likely to be sensitive to deformations and this should be properly specified in the contract documents for a project. The inherent variability of properties should be taken into account using statistical procedures.

2.3 Value of α_{cc} [BS EN 1992-1-1:2004, 3.1.6 (1)P]

Design compressive strength f_{cd} is defined in BS EN 1992-1-1:2004, 3.1.6 (1)P as:

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$$

where

f_{ck} is the characteristic strength of concrete;

γ_c is the partial factor; and

α_{cc} is a coefficient;

which according to BS EN 1992-1-1:2004 takes into account the long-term effects on the compressive strength and unfavourable effects resulting from the way the load is applied.

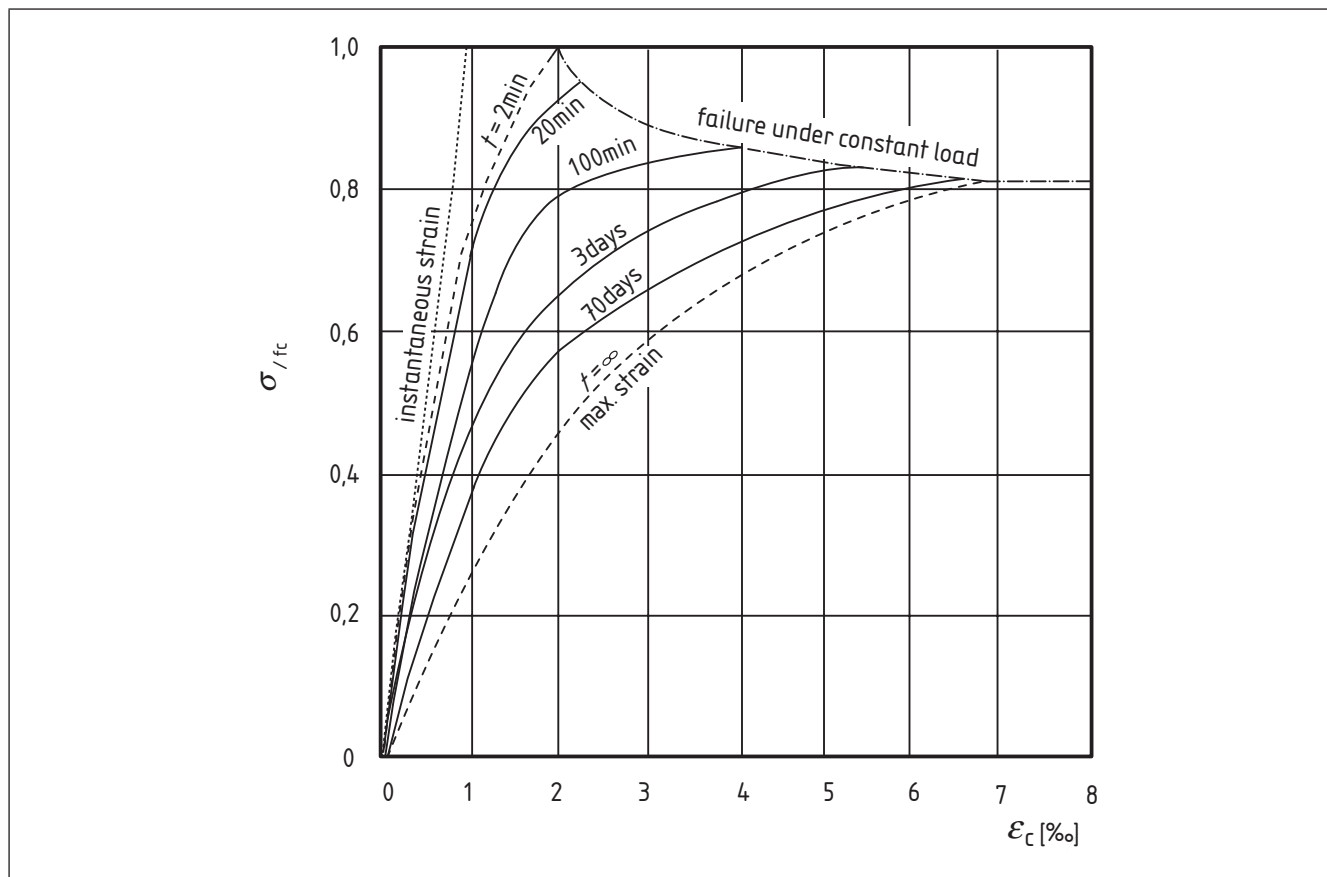
²⁾ Data supplied by reinforcement manufacturers.

BS EN 1992-1-1:2004 recommends a value of 1,00 for α_{cc} but NA to BS EN 1992-1-1:2004 recommends changing this to 0,85, which was the value used in DD ENV 1992-1-1.

During the development of BS EN 1992-1-1:2004 the following three reasons were given for recommending a value of 1,00 for α_{cc} .

- Based on tests by Rüsch (1960s) [3] it is stated that the stress at failure under constant load depends on the duration of loading. This approaches 80% of the short-term capacity as the duration of loading increases and is higher for shorter duration (see Figure 2).
- The strength at 28 days forms the basis for design calculations. However, in practice when structures receive their design loads they will be older than 28 days and there is an increase in strength of the order of 12% for concrete with normal hardening cement. Therefore a considerable part of the sustained loading effect has already been compensated.
- The resistance equations in design codes such as BS EN 1992-1-1:2004 are derived from laboratory tests with a duration of about 90 min, when the strength of concrete would have already fallen by about 15% compared to the short-term value. Therefore part of the effects for sustained loading has been already built in the resistance equations.

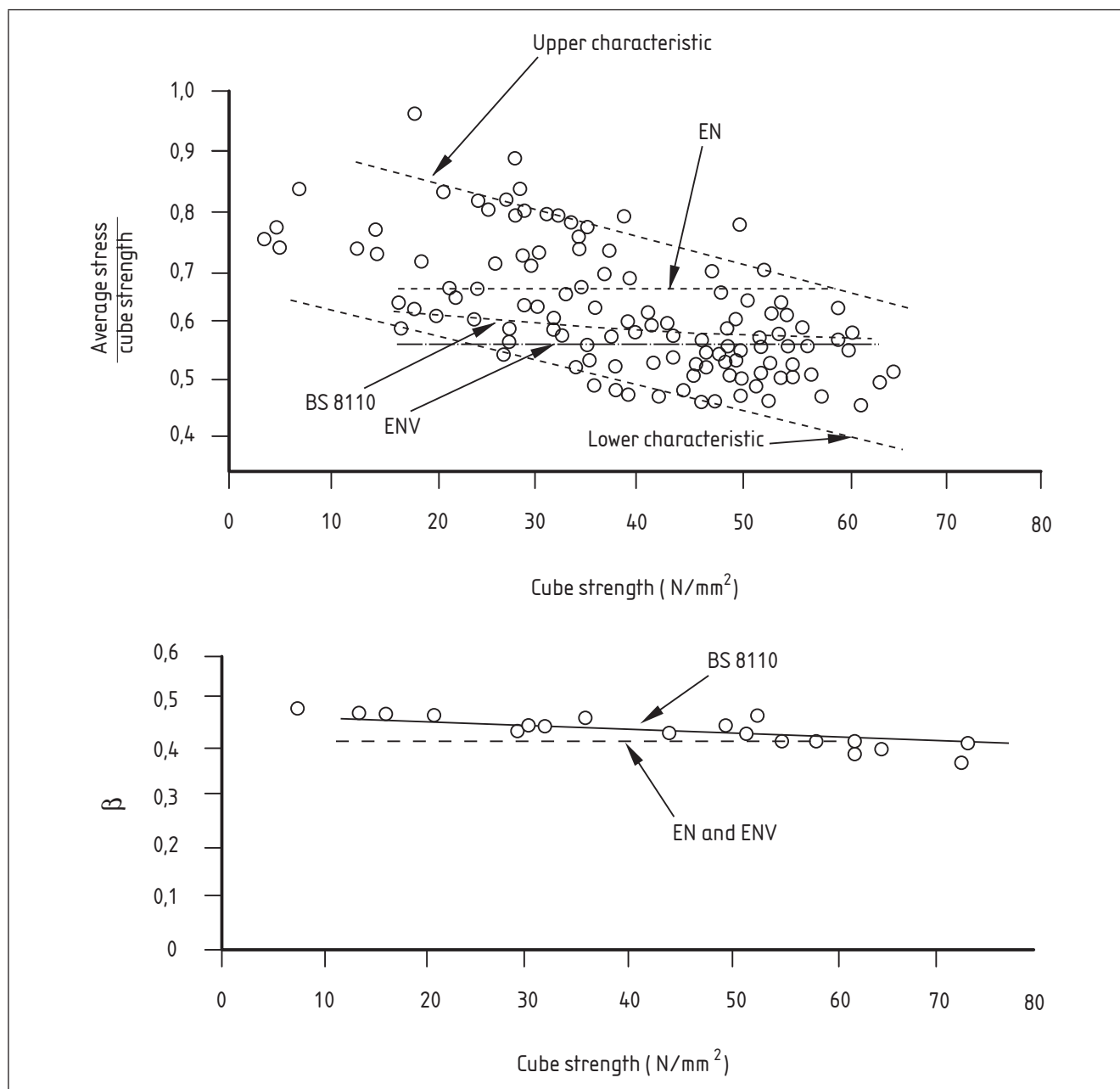
Figure 2 Stress-strain relationship for axial compression for different durations of loading³⁾



³⁾ Image originally sourced from Rüsch [3].

While the UK does not totally reject the arguments given in a), b) and c), it considers that the comparison of the predicted and experimental compression zone behaviour has been ignored. Figure 3 shows the results obtained by Imperial College and the Prestressed Concrete Association for the average stress in compression zone as a fraction of the cube strength of concrete [4]. At higher levels of strength, DD ENV 1992-1-1 (which recommended 0,85 for α_{cc}) and BS 8110-1:1997 gave something approaching a median prediction and BS EN 1992-1-1:2004 (which recommends 1,0 for α_{cc}) is close to the upper characteristic value.

Figure 3 Average stress in compression zone and location of the centroid of the compressive force⁴⁾

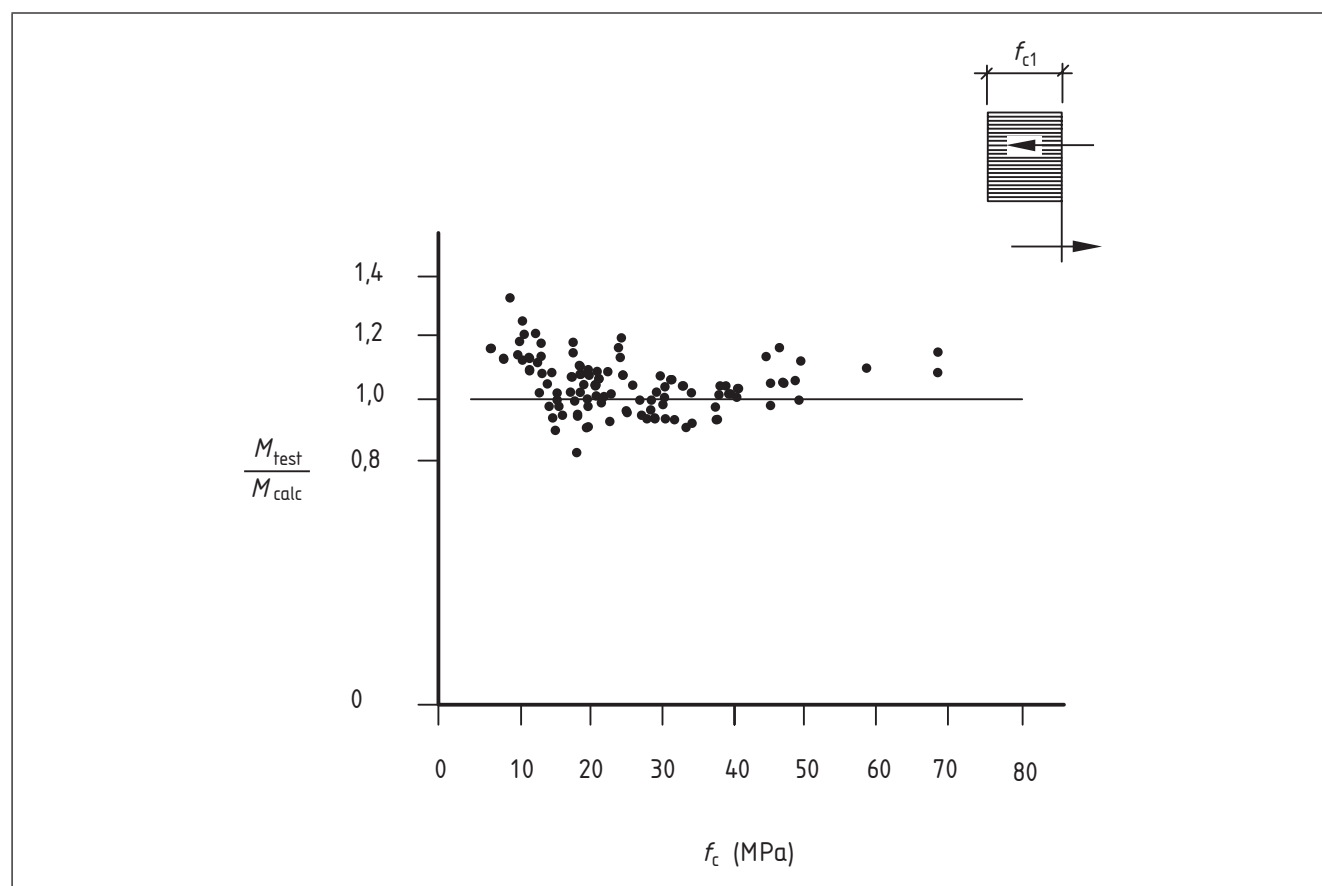


⁴⁾ Image taken from Beeby [5], obtained from tests carried out at PCA and Imperial College [4].

Figure 4 and Figure 5 reproduce the comparisons of test and calculated values from the International Federation for Structural Concrete (*fib*) bulletin, *Structural Concrete – Text book on behaviour, design and performance* [6]. The figures are calibrations for the proposals in the CEB/FIP Model Code 90 [7]. A value of 0,85 for α_{cc} is built into the model code equations. While these figures show that the model code is conservative for low strengths of concrete, the model code provisions give something close to a “best fit” to the experimental results for higher strengths of concrete. If the factor of 0,85 were removed from the provisions the calculations would predict something close to an upper bound to the data. It seems that α_{cc} is a necessary calibration factor to permit the BS EN 1992-1-1:2004 provisions to predict a reasonable mean value for the stress in compression zone for short-term tests.

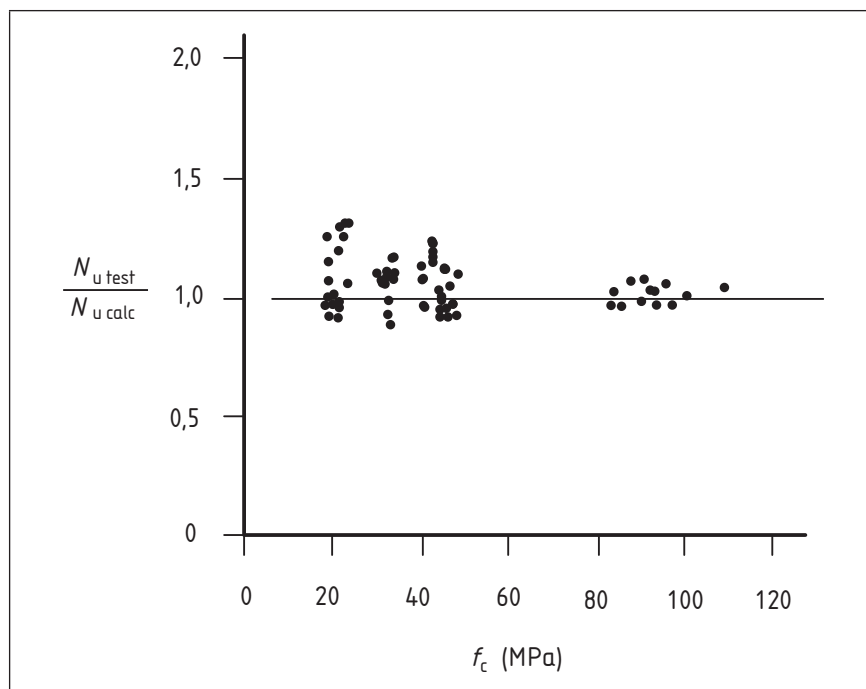
Therefore, a value of 1,0 for α_{cc} is not currently justified in the absence of a proper calibration of BS EN 1992-1-1 provisions against up-to-date test data. It should also be noted that any value other than 0,85 will be inconsistent with BS EN 1994-1-1.

Figure 4 Comparison of experimental and calculated ultimate moments for over-reinforced beams⁵⁾



⁵⁾ Image taken from *Structural Concrete – Textbook on behaviour, design and performance* published by *fib* [6].

Figure 5 Comparison of experimental and calculated strengths of axially loaded columns⁶⁾ – Parabolic rectangular diagram for a maximum stress of $0,85f_c$ and a maximum strain of 0,002



Strictly speaking the factor α_{cc} needs to be applied only to concrete compressive strengths related to stress block used for flexure and axial loading including strut and tie models and bearing stresses in the connections for precast elements. Unfortunately, the parameter f_{cd} , as defined including α_{cc} occurs in the resistance models for shear, torsion and web compression (i.e. BS EN 1992-1-1:2004, 6.2, 6.3 and 6.4). While there will be no practical consequence if the recommended value of 1,0 had been accepted for α_{cc} , complications in use arise when any other value is chosen. The NA to BS EN 1992-1-1:2004 recommends two values for α_{cc} , namely 0,85 for flexure and axial loading and 1,0 for all other phenomena. However the designer can use the conservative value of 0,85 for all phenomena for pragmatic reasons of minimizing the scope for errors.

2.4 Value of α_{ct} [BS EN 1992-1-1:2004, 3.1.6 (2)P]

Design tensile strength f_{ctd} is defined in BS EN 1992-1-1:2004, 3.1.6 (2)P as:

$$f_{ctd} = \alpha_{ct} f_{ctk, 0,05} / \gamma_c$$

where

$f_{ctk, 0,05}$ is the characteristic tensile strength of concrete (5% fractile);

γ_c is the partial factor; and

α_{ct} is a coefficient.

BS EN 1992-1-1:2004 recommends a value of 1,0 for α_{ct} and the NA to BS EN 1992-1-1:2004 adopts this recommendation.

⁶⁾ Image taken from *Structural Concrete – Textbook on behaviour, design and performance* published by fib [6].

Data for behaviour of concrete in tension, unlike that for compression zones, is limited. However, there is an indirect justification for the value recommended in BS EN 1992-1-1:2004. Bond stress is expressed as a function of tensile strength f_{ctd} . The design values for the ultimate bond stress f_{bd} in BS EN 1992-1-1:2004 compare better with those in BS 8110-1:1997 if α_{ct} is taken as 1,0 rather than 0,85 (the value recommended for α_{cc} in NA to BS EN 1992-1-1:2004). The design bond stresses will be unduly conservative if 0,85 is used for α_{ct} with consequent increases in lap and anchorage lengths.

2.5 Reinforcement type [BS EN 1992-1-1:2004, 3.2.2 (2)P]

All the rules and properties for steel reinforcement in BS EN 1992-1-1:2004, 3.2 apply to ribbed and weldable reinforcement.

There is no technical reason why BS EN 1992-1-1:2004 cannot be used in conjunction with other types of reinforcement, provided allowance is made for their properties and behaviour. Matters to consider include:

- a) the value of yield strength range f_{yk} , which in turn affects the calculation of minimum reinforcement for control of cracking and the calculation of deflections;
- b) the effect of bond properties on the calculation of crack spacing, which is determined by BS EN 1992-1-1:2004, Expression (7.11);
- c) the effect on ultimate bond stress, e.g. for plain bars BS EN 1992-1-1:2004, Expression (8.2) for bond stress should be replaced by:

$$f_{bd} = \eta_1 \eta_2 (0,36 \sqrt{f_{ck}}) / \gamma_c$$

where

η_1 and η_2 are as defined in BS EN 1992-1-1:2004, 8.4.2 (2); and

- d) the minimum diameter of mandrel for bends, hooks and loops.

The mandrel diameter for plain bars should be 4ϕ .

When stainless steel or coated reinforcement is used, design parameters recommended in authoritative documents (e.g. Concrete Society Technical Report No. 51 [8]) should be adopted. There have been recent developments in stainless steel (e.g. higher strength). The designer should satisfy himself that the literature is appropriate the grade of reinforcement to be used.

2.6 Welding of reinforcement bars [BS EN 1992-1-1:2004, 3.2.5 (2)P]

Welding is required to be carried out in accordance with BS EN ISO 17660. This standard does not fully address the needs of the UK construction industry and reference should instead be made to the National Structural Concrete Specification [9].

2.7 Cover to reinforcement for durability [BS EN 1992-1-1:2004, 4.4.1.2 (5)]

NA to BS EN 1992-1-1:2004 recommends using BS 8500-1:2006, Tables A.4, A.5 and A.11 for the selection of concrete composition and cover to reinforcement for different exposure classes instead of BS EN 1992-1-1:2004, Tables 4.3N, 4.4N and 4.5N.

NA to BS EN 1992-1-1:2004 sets a value of zero for the additive safety element, $\Delta c_{dur,\gamma}$. However, the designer can increase the cover to reinforcement by setting a higher value of $\Delta c_{dur,\gamma}$ for particular exposure classes where it is considered that additional safety is required. For example $\Delta c_{dur,\gamma}$ can be 10 mm for pre-stressed members in a chloride environment.

NOTE 1 BS 8500-1:2006 and BS 8500-2:2006 are to be used in the UK when specifying concrete to BS EN 206-1.

2.8 Allowance in design for deviation in cover to reinforcement [BS EN 1992-1-1:2004, 4.4.1.3]

The provisions in BS EN 1992-1-1:2004, 4.4.1.3 for deviation in cover are logical. Bearing in mind the durability problems caused by incorrect cover, BS EN 1992-1-1:2004 warns that deviations in cover will occur in practice and states that an allowance for this should be made in design.

BS EN 1992-1-1:2004, 4.4.1.3 requires an allowance (Δc_{dev}) to be made in *design* for the possible deviation of cover and allows the values for deviation in cover to be chosen from the following.

- a) 10 mm should be added to the minimum cover unless the specific actions described in b) and c) are taken.
- b) When concrete covers are monitored by an inspection regime as part of a recognized quality scheme (e.g. as required for members of SpeCC)⁷⁾ there is scope to reduce the additional cover down to 5 mm.
- c) When the non-complying structural members are rejected the additional cover can be reduced to zero. This can generally be applied only to factory produced components operating checking procedures.

The negative deviation on cover noted in project specifications should be limited to 5 mm for a) and b) so that the absolute minimum cover is always achieved.

2.9 Simplified load combinations [BS EN 1992-1-1:2004, 5.1.3 (1)P]

2.9.1 General

BS EN 1992-1-1:2004, 5.1.3 (1)P recommends the classical load patterns for the analysis of continuous members, namely, variable load on alternate spans and adjacent spans. The NA to BS EN 1992-1-1:2004, while permitting this, also provides:

- a) for beams and slabs, separate cases of all spans loaded and alternate spans loaded; and
- b) for slabs only, under the conditions specified in the UK National Annex (which covers a large majority of cases in practice) only a single load case of all spans loaded.

⁷⁾ The registration scheme for Specialist Concrete Contractors (SpeCC), www.speCC.co.uk.

In cases where “all spans loaded” alone is used for slabs, the support moments should be reduced by 20% and the span moments adjusted correspondingly.

NA to BS EN 1992-1-1:2004 notes that the simplified load arrangements can be used with the load combination expressions in BS EN 1990:2002+A1:2005, namely either Expression (6.10) or the more adverse of Expressions (6.10a) or (6.10b). If Expression (6.10a) or (6.10b) is used, each span should be tested separately to determine which of the two expressions applies.

2.9.2 Comparison of BS EN 1992-1-1:2004 and BS 8110

The additional load arrangements recommended in NA to BS EN 1992-1-1:2004 are similar to those in BS 8110-1:1997. The one difference lies in the magnitude of the permanent load in the spans not loaded with the variable load.

BS EN 1992-1-1:2004, 2.4.3 recommends that for each permanent action either the lower ($\gamma_{G,inf}G_k$) or the upper ($\gamma_{G,sup}G_k$) design value (whichever gives the more unfavourable effect) should be applied throughout the structure. It also notes that there may be exceptions, e.g. verification of static equilibrium.

In contrast, BS 8110 (all parts) uses $1.0G_k$ on spans assumed to be carrying the minimum design load and $1.4G_k$ on spans assumed to be carrying the maximum design load. The implications of this difference have been reported by Narayanan [10]. The position of the points of contraflexure and the magnitude of the bending moment, in cases where there are hogging moments over the whole span, are affected by the choice of load patterns. While there will be no practical consequence for beams, the top reinforcement will need to be carried further into spans in some slabs.

2.9.3 Justification of the single load case

2.9.3.1 General

The background to the single load case in BS 8110-1:1997 is given by Beeby [11]. The issues discussed in that report are still relevant and the report should be consulted for details.

The report justifies the choice of the single load case mainly on probabilistic grounds. It also points out that a number of features in the behaviour of slab systems are not taken into account in design and they provide a margin of safety against any unconservative approximations in the choice of the load pattern. An outline of the report is summarized 2.9.3.2, 2.9.3.3 and 2.9.3.4.

2.9.3.2 Probabilistic considerations related to load patterns

The classical load patterns are deterministic in their search for worst critical moments. They do not necessarily correspond to physical reality or imply any specific probability of occurrence.

For instance, if the loading in one span is assumed to be statistically independent of loading in other spans, the arrangement of loads (the joint probability of occurrence of which is the same as the probability of occurrence of the single load) is likely to be $\gamma_Q Q_k$ on one span and $\psi \gamma_{Qi} Q_{ki}$ in all other spans. The reduction factor ψ accounts for the joint probability.