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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/mm^2 [the characteristic value specified in the now withdrawn British Standard for reinforcement BS 4449:1997 [1)]] 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 + 2,5σ_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/mm^2. 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/mm^2. 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 reinforcement**[2)]
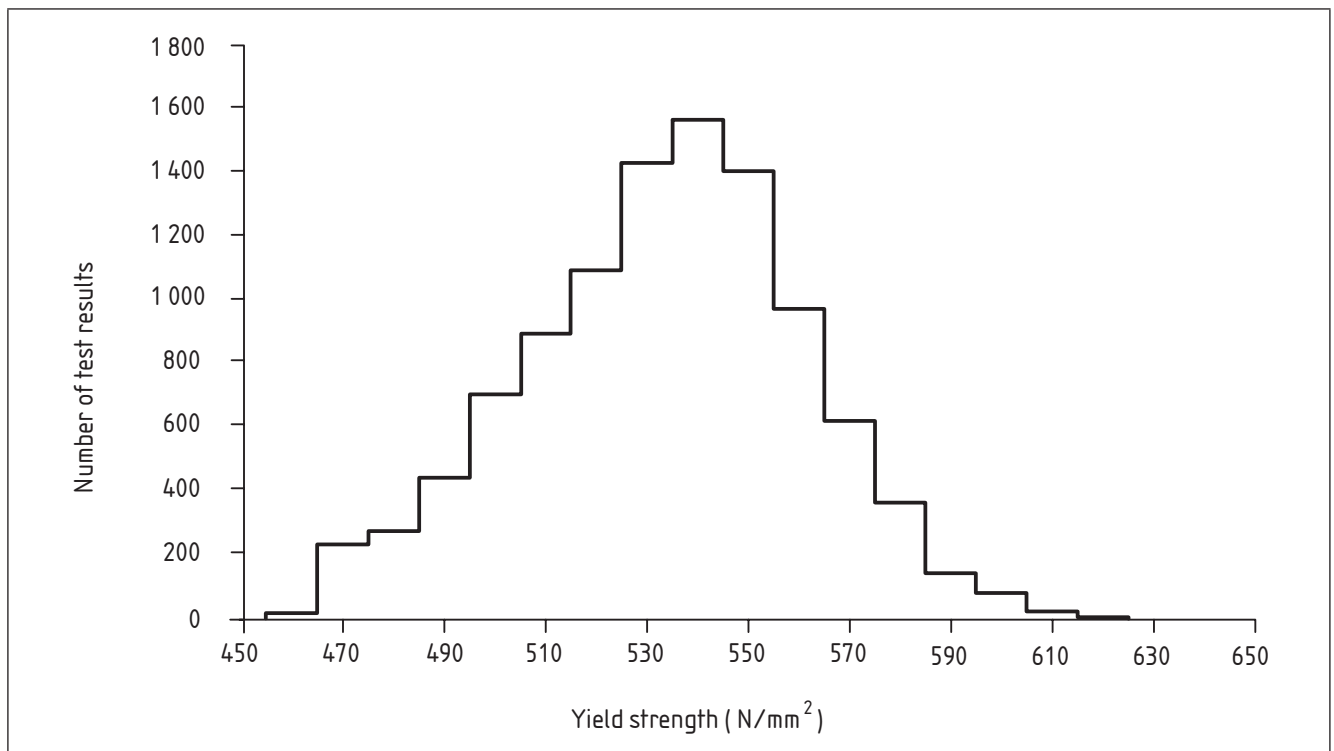
2.2 Elastic deformation properties of concrete [BS EN 1992-1-1:2004, 3.1.3]

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}.

a) 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).

b) 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.

c) 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 loading** [3]

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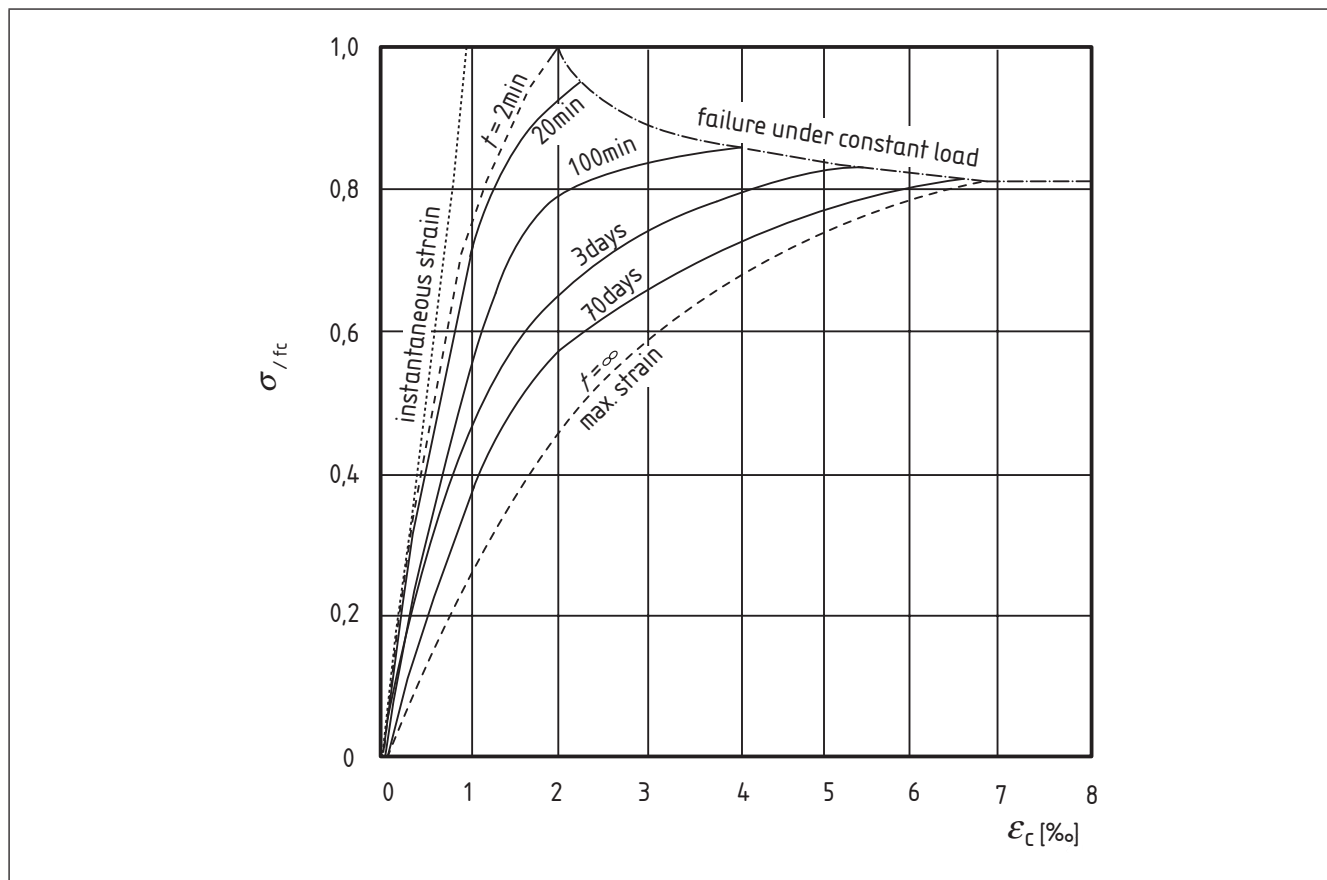
[3]   Image originally sourced from Rüsch [3].

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 force**[4]

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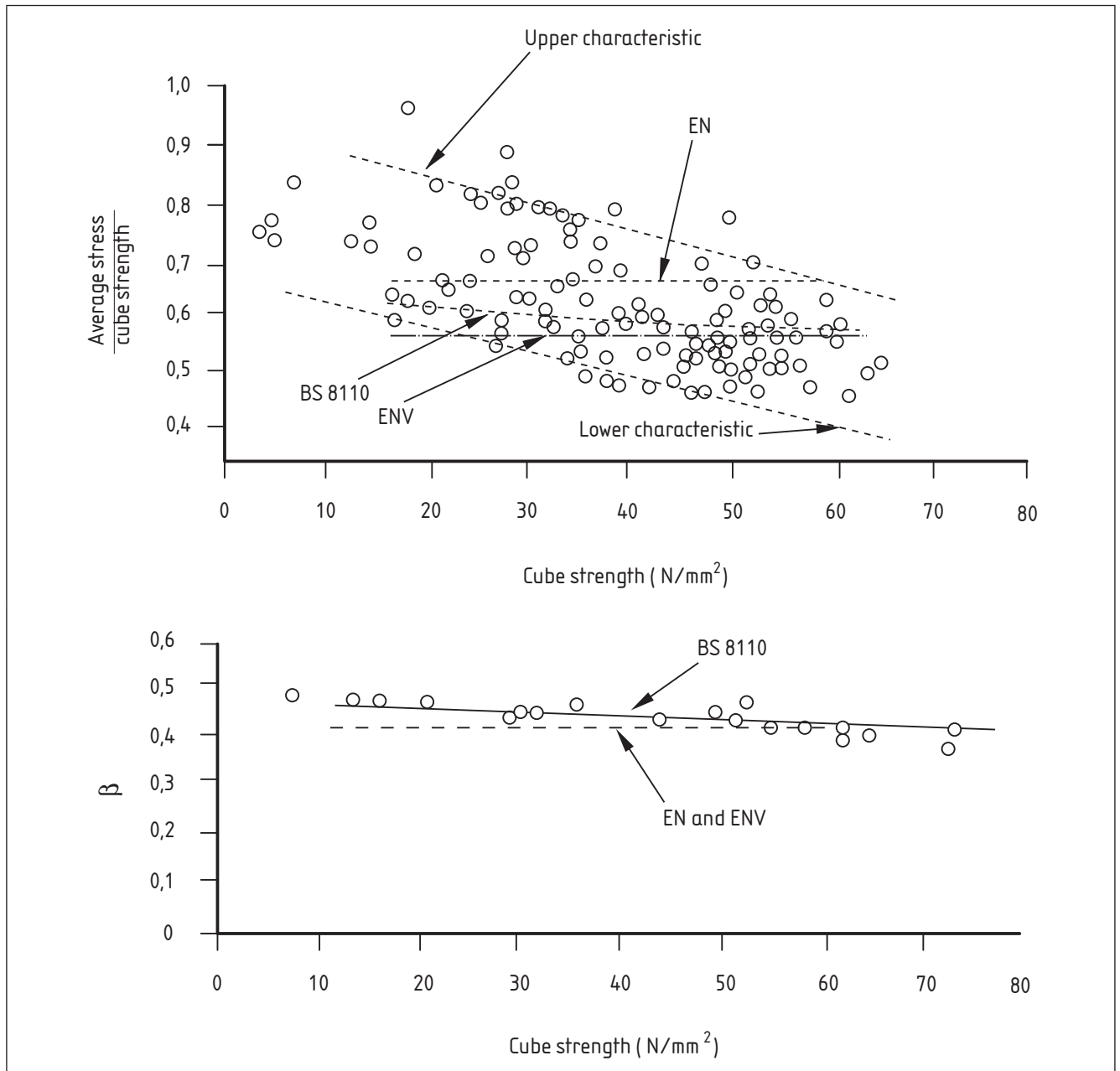
[4]   Image taken from Beeby [5], obtained from tests carried out at PCA and Imperial College [4].

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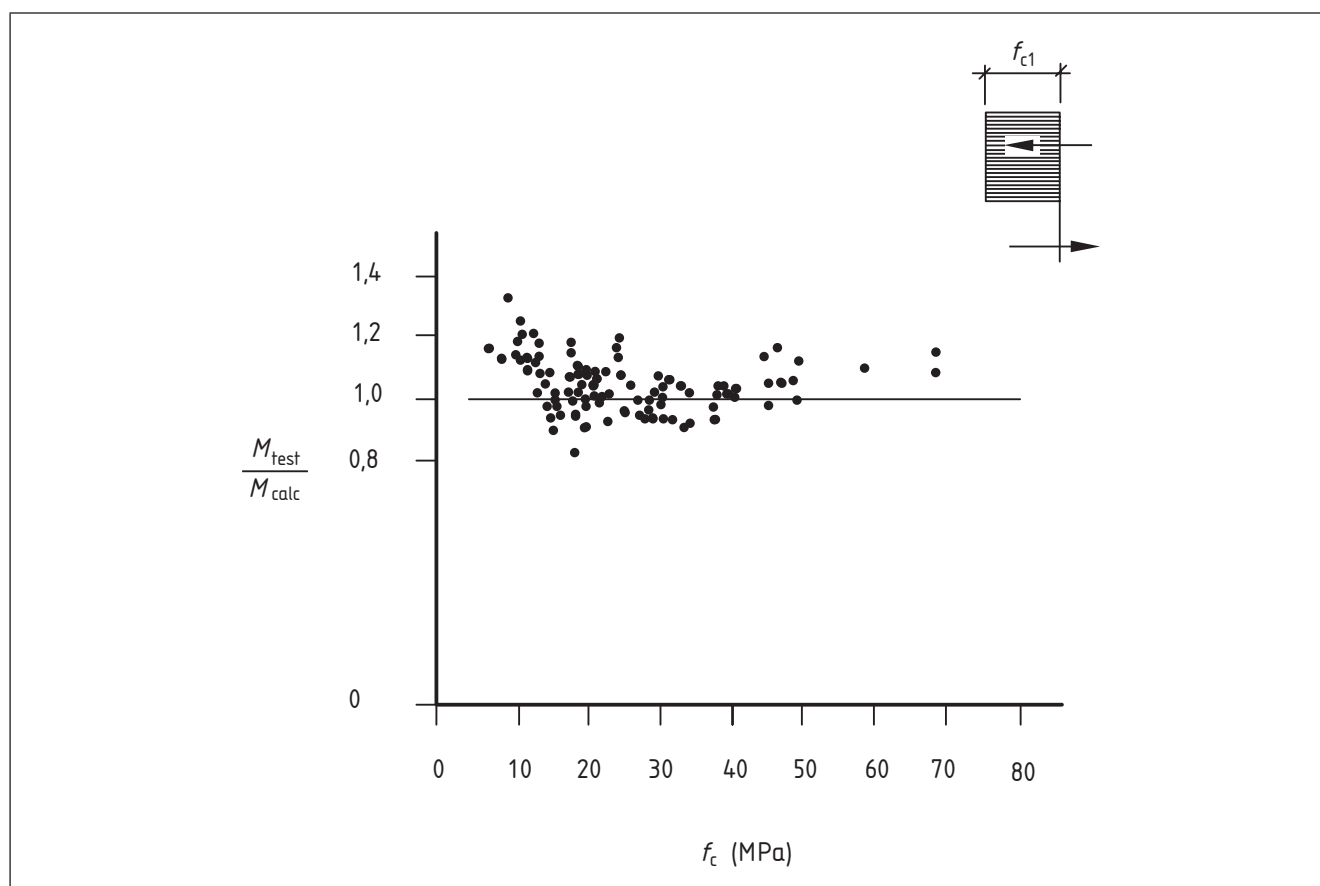
[5)]   Image taken from *Structural Concrete – Textbook on behaviour, design and performance* published by *fib* [6].

Figure 5 **Comparison of experimental and calculated strengths of axially loaded columns[6)] – Parabolic rectangular diagram for a maximum stress of 0,85f_c and a maximum strain of 0,002**
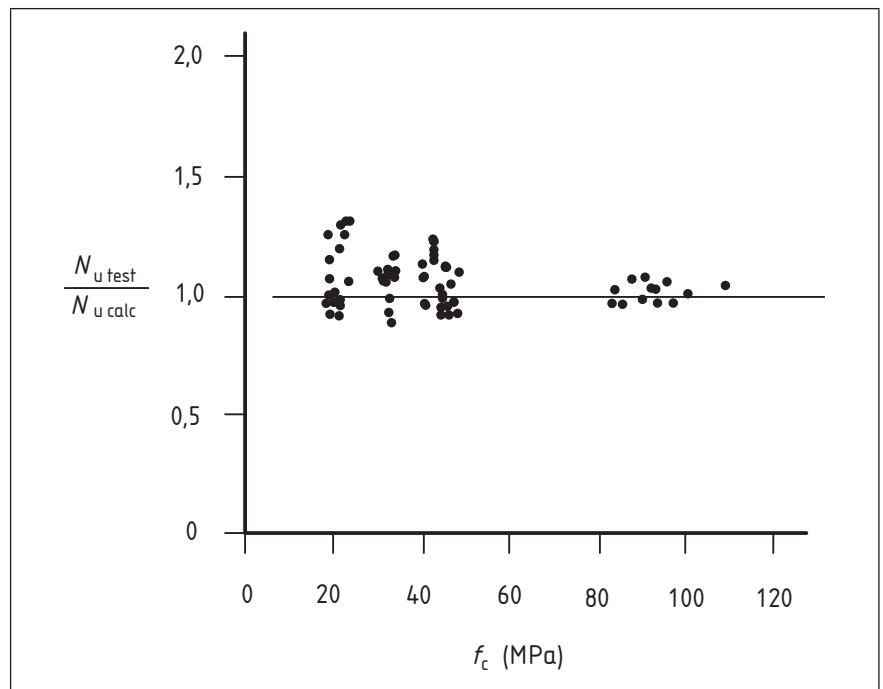
Strictly speaking the factor α_{cc} needs to be applied only to concrete compressive strengths related to stress block used for flexure and axial loading including strut and tie models and bearing stresses in the connections for precast elements. Unfortunately, the parameter $f_{cd,}$ as defined including 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.

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[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.

If on the other hand it is assumed that the loads on all the spans are fully correlated, the critical load case is the single load case with the maximum load on all spans.

For the classical load patterns to be true there has to be a negative correlation between loading on one span and that on the others, i.e. when one span has the maximum load, there is more than a random chance that the other spans are unloaded. Evidence exists to suggest otherwise, for example BRE Current Paper 3/71 [12]. This BRE paper implies that the correct probabilistic envelope for design lies between "all spans loaded" case and the case with maximum load on one span with the most probable loads on other spans. The bending moments from the all spans loaded case with 20% downward redistribution is regarded as a good approximation.

In the case of floors subject to office loading there is a further hidden factor of safety. The NA to BS EN 1992-1-1:2004 requires the area of the slab bay to be at least 30 m^2 for the single load case to be applied. Surveys of office floors indicate that the characteristic load corresponding to this area is in practice 2,0 kN/m^2 rather than the 2,5 kN/m^2. The latter corresponds to a floor area of 15 m^2. Therefore all floor bays in excess of 15 m^2 have an additional margin of safety.

More recently Beeby and Fathibitaraf [13] have argued that the only practically possible failure mechanism is the simplest possible mechanism. This is a failure that occurs within one span. They point out that no viable failure mechanism can be postulated for the adjacent spans loaded case. For the alternate spans loaded case, not only do sagging hinges need to form in the loaded span but also hogging hinges in the unloaded span. Further, hinges have to form in walls and columns above and below the slab, where these elements are monolithic. It is argued that the failure mechanisms implicit in the classical load patterns are inherently so unlikely that they can be ignored in practical design.

2.9.3.3 Serviceability considerations

The deflection of a continuous member can be shown to be dependent more on the amount of reinforcement provided for sagging moment in the span than on that provided over the supports.

In the classical load arrangements the alternate spans loaded case defines the maximum midspan moment. The 20% downward redistribution of support moments in the all spans loaded case generally resulted in span bending moments similar to the classical load cases. The reduction in the support moments can affect the cracking over supports but cracking in slabs is rarely a problem, provided the steel remains below yield.

Overall, the use of the single load case with 20% redistribution gives a serviceability performance which was not significantly different to that obtained using the classical load arrangements.

2.9.3.4 Behaviour of slab systems at ultimate limit state

2.9.3.4.1 Membrane effects

Membrane effects arise where a slab is restrained from outward movement once yield has occurred. As a result the load bearing capacity of the slabs can be increased very substantially. There are practical difficulties in quantifying the degree of restraint and in characterizing the likelihood of the restraint being present.

Nevertheless it seems reasonable to assume that there are sufficient slab surrounding any incipient local flexural failure to provide enough restraint to ensure that local failure is not a critical failure mechanism.

The critical failure can be expected to be one that requires yield lines across the whole of a slab panel.

The minimum bay size of 30 m² stipulated for the application of the single load case will ensures that there is a margin of safety of some 15% to 20% available to account for any unconservative approximations.

2.9.3.4.2 Probabilistic considerations affecting the strength of yield mechanisms

If it is accepted that the critical failure mechanism is one which takes in the whole of a panel, the length of yield line making up this mechanism will be large involving a substantial number of reinforcement bars. Common sense suggests that the probability of all this length and all the bars being at their design strength is very small. Therefore there will be a further margin of safety here although it is difficult to establish what it might be.

The standard deviation of reinforcement strength used in defining the partial safety factor is made up of a number of sources of variability, namely, "between mills", "between castings", "between billets" and "between bars". If all the bars in one area are the same size, they are likely to be from the same billet and the variability is then only "between bars", which is considered to be the smallest contributor to the overall variability of reinforcement strength. Although not large, there is some extra safety as the number of sources of variability is reduced to three in this case.

2.9.3.5 Summary

In summary, the additional choices in NA to BS EN 1992-1-1:2004 for load arrangements are based on detailed studies and have been successfully used in the UK for about 20 years.

2.9.4 Upper and lower characteristic values of permanent loads

In the context of load combinations, it is appropriate to comment on the use of upper and lower characteristic values of permanent loads ($G_{k,sup}$ and $G_{k,inf}$) noted in BS EN 1990:2002+A1:2005. This distinction is required only if the variability of the value of G cannot be considered small (coefficient of variation greater than 10%). Most practical structures do not fall into this category and a single load case using the mean value of permanent loads is adequate.

BS EN 1990:2002+A1:2005 also adds that in a structure, which is likely to be sensitive to variation of permanent action, $G_{k,sup}$ and $G_{k,inf}$ should be considered even if the variability in the permanent action is small.

2.10 Redistribution of bending moments [BS EN 1992-1-1:2004, 5.5]

The formulae in NA to BS EN 1992-1-1:2004, **5.5** are similar to those given in the Concrete Society report on high strength concrete [14]. The parameters have been chosen on the assumption that the reinforcement conforms to BS 4449:2005, which uses a yield strength of 500 MPa. When steels with higher yield strength are used, more restrictive values should be applied to ensure that reinforcement yields prior to the concrete reaching ultimate strain.

2.11 Calculation of effective length of columns [BS EN 1992-1-1:2004, 5.8.3.1, 5.8.3.2 (4) and (5)]

2.11.1 General

BS EN 1992-1-1:2004, **5.8.3.2** (4) and (5) might present some difficulties of interpretation. These relate to:

a) the need to include the stiffness of columns attached to the column under consideration;

b) the method of allowing for cracking in the calculation of the stiffness of the restraining members; and

c) the curvature of bending to be assumed in columns.

2.11.2 Effective length of columns

If all the columns in a frame are assumed to be close to failure at the same time it is necessary to include the stiffness of the columns attached to the one under consideration. The resulting effective lengths will be considerably greater than those currently used in the UK. However it is considered that such an assumption is unduly over cautious, particularly in structures in which the stiffness of columns does not vary significantly.

When considering the buckling of a column it is more realistic to attribute the design ultimate material properties to this column and average material properties to the rest of the structure. Similarly, the accidental eccentricity should be assumed to apply only to the failing column. Under these conditions the deformation of the non-failing columns will be negligible and will not contribute to the flexibility of the node of the column being designed. For these reasons it is satisfactory to ignore the attached non-failing columns in the consideration of the effective length of a column. It is then necessary only to consider the rotational restraint offered to the ends of the columns by the attached beams at each end.

Contribution to (θ/M) of beams that might be cracked may be conveniently taken as $(l/2EI)_b$ rather than calculating the cracked properties.

One further point to note is that the expressions for effective lengths use relative flexibilities k_1 and k_2 at end 1 and 2 of the column [i.e. $k = (\theta/M)(EI/l)$]. In this expression (EI/l) refers to the column properties. The expression may be rewritten as $[(EI/l)_c]/(\theta/M)$, i.e. stiffness of the column under consideration $[(EI/l)_c]$ divided by the sum of the stiffness of the beams $[(\Sigma 2EI/l)_{beams}]$ attached to the column.

In summary, in considering the effective length of a column in regular structures in which the stiffness of adjacent columns do not vary significantly (say, difference not exceeding 15% of the higher stiffness), the relative flexibility of each end of the column should be calculated ignoring the contribution of the attached columns, which are assumed to be non-failing at the same time. In structures where the stiffness of adjacent columns varies significantly, the stiffness of the adjacent column should be taken into account in the calculation of the flexibility of the resisting elements.

At each end of the column, the stiffness of each beam attached to it should be modelled as $2(EI/l)_{beam}$ to allow for the effect of cracking, and then summed over all the attached beams.

Calibrations suggest that the procedure described leads to effective lengths similar to those tabulated in BS 8110-1:1997

2.11.3   Calculation of limiting slenderness ratio, λ_{lim}

In regular braced structures in which adjacent spans of beams do not differ by more than 15%, columns may be assumed to be in double curvature bending for the calculation of λ_{lim} (i.e. value of moment ratio $r_m < 0$). Conditions causing single curvature bending are unlikely to be the critical governing condition for the design of such columns.

2.12   Design moment in columns [BS EN 1992-1-1:2004, 5.8.7.3 and 5.8.8.2]

BS EN 1992-1-1:2004, **5.8.7.3** and **5.8.8.2** are concerned primarily with the calculation of the nominal second order moment M_2 in slender columns. The method given in BS EN 1992-1-1:2004 for calculating the design moment M_{Ed} by combining M_2 with the first order moments applies to mid-heights of columns in braced structures. The value of M_{Ed} thus obtained may not be the largest in the column. It is left to the designer to decide on the value of the controlling moment to be combined with the axial load in design.

For braced structures:

   M_{Ed} = maximum of $(M_{0e} + M_2)$, (M_{02}) or $(M_{01} + 0{,}5 M_2)$.

For unbraced structures:

   $M_{Ed} = (M_{02} + M_2)$;

where

   M_{0e} is the equivalent moment defined in BS EN 1992-1-1:2004;

   M_2 is the second order moment;

   M_{01} and M_{02} are the first order moments at the ends of the column including the effect of imperfections. Numerically $M_{02} > M_{01}$.

2.13   Effect of prestressing at ultimate limit state [BS EN 1992-1-1:2004, 5.10.8]

The allowance of 100 N/mm^2 in BS EN 1992-1-1:2004, **5.10.8**(2) for the increase in stress from the effective prestress to the stress at ultimate limit state is conservative for tendons in typical beams and slabs, particularly for mid-span sections. However, it is not safe when the eccentricity of the tendons is small. Therefore, NA to BS EN 1992-1-1:2004 imposes some additional limitations.

BS EN 1992-1-1:2004, **5.10.8**(3) assumes that the critical sections of a structural member are analysed in the conventional way to resist the forces given by structural analysis. However, if the ultimate strength is determined directly using the non-linear analysis permitted in BS EN 1992-1-1:2004, **5.7** and **5.8.6**, there needs to be consistency in the specification of forces and material properties used for the structural analysis and the section analysis.

2.14 ## Design shear – Point loads close to support [BS EN 1992-1-1:2004, 6.2.2 (6)]

The rules for design shear are written for the case of point loads applied to the top surface of one-way spanning elements (e.g. beams and slabs). It is more restrictive than current UK practice. Alternative approaches might be required for other cases (e.g. when upward and downward loads are applied to the structure or when the element is designed for an envelope of shear obtained from a number of load cases). See also Jackson et al [15].

Where a point load is located at a distance a_v from the support such that $0,5d \leq a_v \leq 2d$ from the face of the support BS EN 1992-1-1:2004, **6.2.2**(6) recommends that for shear design a reduced value of applied shear, βV_{Ed} is considered, where $\beta = a_v/2d$. No special indication is noted as regards the angle of the strut that should be used to calculate $V_{Rd,max}$. Generally in practical cases point loads close to support will need to be considered in conjunction with other loads on the member. Design shear V_{Ed} between the point load and the support is $V_{Ed} = V_{Ed,other} + \beta V_{Ed,pointload}$. The first term is the shear resulting from all loads other than the point load close to support.

Angle θ_P is defined as that between the longitudinal axis of the member and a line joining the face of the support and the point load. The two following cases need consideration:.

a) $\theta_P \leq 45°$

   Calculate $V_{Rd,max}$ using $\cot \theta = \cot\theta_P$ in BS EN 1992-1-1:2004, Expression (6.9). If $V_{Rd,max} \geq V_{Ed}$, the shear reinforcement required should be placed in the central $0,75a_v$ such that $V_{Ed} \leq A_{sw}f_{ywd}\sin\alpha$, where α is the inclination of the shear reinforcement to the longitudinal axis of the member as described in BS EN 1992-1-1:2004, **6.2.3**(8).

   If $V_{Rd,max}$ using $\cot\theta_P$ in BS EN 1992-1-1:2004, Expression (6.9) is $< V_{Ed}$, calculate $\cot\theta$ by equating V_{Ed} to V_{RDmax} from BS EN 1992-1-1:2004, Expression (6.9). Using this value of $\cot\theta$, calculate the shear reinforcement for the full design shear force (i.e. without applying β factor) using BS EN 1992-1-1:2004, Expression (6.8).

b) $\theta_P \geq 45°$

   In these cases $V_{Rd,max}$ using $\cot\theta = 1,0$ should be calculated and checked that it is less than the design shear V_{Ed}. Shear reinforcement should then be designed in accordance with BS EN 1992-1-1:2004, **6.2.3**(8) and placed in the central $0,75a_v$. Attention is drawn to the requirement that the full shear force (i.e. without applying β factor) should satisfy BS EN 1992-1-1:2004, Expression (6.5).

When $a_v < 0,5d$, reference should be made to replacement Annex B of this Published Document.

2.15 ## Maximum shear resistance [BS EN 1992-1-1:2004, 6.2.3 (3)]

Compared with past practice in the UK (and with other major international codes of practice), the recommended values of maximum shear resistance in BS EN 1992-1-1:2004 can result in significantly high values for the maximum shear force that can be sustained by a member.

The Highways Agency commissioned an investigation to study this (see Jackson and Salim [16]). Available test results were studied to form a view on the maximum shear resistance, effect of the slenderness of the web and the contribution of inclined links. Additional rules have been introduced in NA to BS EN 1992-1-1:2004 as a result of this study but in practice it is anticipated that only a small proportion of cases will be affected by these additional rules.

The study showed that the rules for maximum shear resistance were generally satisfactory compared with tests but there were a few results with excessively low margins. All cases with excessively low margins were found to be governed by the 80% yield rule. Therefore, slightly more conservative rules have been introduced in NA to BS EN 1992-1-1:2004 to rectify this.

The study also found a wide range of cases where there were no test results. Slender webs were a particular concern as some work suggests slenderness reduces crushing stress, but the h/b_w ratio of the most slender webs encountered in practice can be as much as three times that of the most slender webs tested. This issue has been addressed in NA to BS EN 1992-1-1:2004 by applying an overall limit on shear related only to web thickness.

There also appeared to be no adequate tests to justify the substantial benefit for inclined links. This has been addressed in NA to BS EN 1992-1-1:2004 by adding a term related to link angle in the expression for ν_1.

2.16 Basic control perimeter for loaded areas close to edge of slabs [BS EN 1992-1-1:2004, 6.4.2(4)]

When the edge of the slab projects a distance p from the face of the loaded area, basic control perimeter shown in BS EN 1992-1-1:2004, Figure 6.15 should be used when $2p < c_2 + \pi (2d)$, where c_2 is the dimension of the loaded area parallel to the edge. Otherwise the failure will be like that of an internal column and the perimeter shown in BS EN 1992-1-1:2004, Figure 6.13 should be used.

The β factor for circular edge and corner columns may be calculated on the basis of a square column that encloses the circular shape.

2.17 The value of maximum punching resistance adjacent to column [BS EN 1992-1-1:2004, 6.4.5(3)]

In addition to the verification around the column, NA to BS EN 1992-1-1:2004 requires the shear stress at the basic control perimeter to be limited to $2v_{Rdc}$. BCA test results [17] confirm that the level of safety reduces with an increasing contribution from the shear reinforcement and in some cases this could result in unsafe design. Stress limitation at basic control perimeter has been introduced in NA to BS EN 1992-1-1:2004 to guard against this risk. See also [18]. This requirement is not dissimilar to that in the UK National Application Document to DD ENV 1992-1-1.

2.18 Location of punching shear reinforcement with respect to perimeter U_{out} or $U_{out,eff}$ [BS EN 1992-1-1:2004, 6.4.5(4)]

In BS EN 1992-1-1:2004, punching shear reinforcement is calculated at the first perimeter u_1 at 2d from the face of the loaded area/column. The same amount of reinforcement is then provided at each reinforcement perimeter at a spacing of 0.75d. The first perimeter should be located at a distance of between 0.3d and 0.5d from the face of the loaded area/column (see also PD 6687-2:2008).

2.19 Design with strut and tie models [BS EN 1992-1-1:2004, 6.5]

Strut and tie models are generally used when plane sections do not remain plane (i.e. the strain distribution is significantly non linear). This will be the case near concentrated loads, locations where the geometry of the section changes, corners, openings, half joints, etc. These regions are also referred to as discontinuity regions (D regions). Normal beam theory may be applied to sections beyond a distance h from the discontinuity or concentrated load (referred to as undisturbed regions). The dimension h may be approximately taken as the depth of the section in the undisturbed regions. The internal flow of forces in D regions can be described reasonably well using strut and tie method, which is based on the lower bound theorem of plasticity. A set of internal forces in concrete struts and steel ties are found that are in equilibrium with the external loads and without yielding taking place anywhere.

BS EN 1992-1-1:2004, **5.6.4**(1) permits extension of the application of the strut and tie model to undisturbed regions too; but this is not current UK practice. In members without shear reinforcement (e.g. slabs) it may be necessary to include plain concrete ties in the model. If strut and tie modelling is undertaken in members un-reinforced for shear, the shear resistance of sections should be verified using BS EN 1992-1-1:2004, **6.2.2** at all sections where $a_v > 1.5d$, where a_v is the distance of the section from a concentrated load (or support) and d is the effective depth of the section.

2.20 Stress limitation in serviceability limit state [BS EN 1992-1-1:2004, 7.2]

BS EN 1992-1-1:2004, **7.2** states that compressive stress in concrete should be limited to avoid longitudinal cracks, micro-cracks or high levels of creep. It then goes on to explain in very imprecise and unsure language that, in areas exposed to an aggressive environment, it might be necessary to limit the stresses in members that are not provided with confining reinforcement in compression zones. Therefore, BS EN 1992-1-1:2004 would appear to exempt beams and columns because they have transverse links and slabs because flexural compression is rarely if ever a problem.

When considering stress limitation in serviceability it is also helpful to note the following.

a) Stress checks in reinforced concrete members have not been required in the UK for the past 50 years or so and there had been no known adverse effect. Provided that the design has been carried out properly for ultimate limit state there will be no significant effect at serviceability in respect of longitudinal cracking.

b) There has been no evidence either from research or practice that there is a correlation between high compressive stress and durability problems.

c) BS EN 1992-1-1:2004, **7.2**(5) states that the tensile stress in reinforcement should be limited. It does not however state how the stress should be calculated, in particular there is no advice on the basis for the estimation of modular ratio. Rigorous calculations are rather complex, as part of the load will be quasi-permanent. Superposition of the stresses is incorrect, as the addition of short-term load results in upward movement of the neutral axis. In practice it is reasonable to assume that the stress is somewhere between that calculated using short term and long term properties. Use of an intermediate value of 15 for the modular ratio is a reasonable approach. This was recommended in DD ENV 1992-1-1, which preceded BS EN 1992-1-1:2004.

2.21 Crack control [BS EN 1992-1-1:2004, 7.3]

2.21.1 Control of cracking without direct calculation [BS EN 1992-1-1:2004, 7.3.3]

The notes to BS EN 1992-1-1:2004, Table 7.2N and Table 7.3N state the assumptions on which the tables are based. For cases where the assumptions are not met, the results should be considered as approximations. In particularly sensitive cases, it is recommended that crack width is verified using the calculation procedure in BS EN 1992-1-1:2004, **7.3.4**.

2.21.2 Calculation of crack widths [BS EN 1992-1-1:2004, 7.3.4]

While the use of the code provisions will be straightforward in many practical cases, some ambiguities might arise in some cases. Figure 6 illustrates some typical cases. The following guidance may be followed.

a) Effectiveness of a bar to control cracking decreases with increasing distances from the bar.

It will be found that it is convenient to base crack width calculations on a local zone around the reinforcement. In such cases $A_{c,eff}$ for the purpose of calculating $\rho_{p,eff}$ should be as shown in Figure 6a) and should *not* be calculated using BS EN 1992-1-1:2004, **7.3.2**(3).

b) The value of c in BS EN 1992-1-1:2004, Expression (7.11) is the perpendicular distance from the concrete surface to the nearest face of the bar under consideration.

When considering bars in the outer layer c is generally the thickness of nominal cover, c_{nom}, unless cover in excess of c_{nom} is provided in which case c is the thickness of actual cover.

When considering bars in an inner layer where a transverse bar between the bar under consideration and the surface of the concrete is present c is generally the sum of the thickness of nominal cover to the transverse bar and the diameter of transverse bar, unless cover to the transverse bar is in excess of c_{nom}, in which case c is the thickness of the actual cover to transverse bar plus the diameter of transverse bar.

When considering bars in an inner layer, the adjustment to the calculated crack width given in the notes to NA to BS EN 1992-1-1:2004, Table NA.4 should be modified as $(c_{min,dur} + \Delta_{c,dev} + $ size of outer bar)/cover used in BS EN 1992-1-1:2004, Expression (7.11), which is defined above.

In cases where $h_{c,eff}$ (for the purpose of calculating $\rho_{p,eff}$) is less than $(c + 0.5\phi)$, the calculated value of $h_{c,eff}$ may be assumed to be centred about the bar contrary to BS EN 1992-1-1:2004, Figure 7.1 [see Figure 6a)].

When the bar spacing is $\leq 5(c + 0.5\phi)$, BS EN 1992-1-1:2004, Expression (7.8) in conjunction with BS EN 1992-1-1:2004, Expression (7.11) may be used noting that cover dimension c is as described above [see Figure 6a)].

When reinforcement comprises multiple bar sizes and/or spacing, maximum crack width may be related to the bar with the maximum value of $[\phi/\rho_{p,eff}]$.

When the bar spacing exceeds $5(c + 0.5\phi)$, the strain calculated by BS EN 1992-1-1:2004, Expression (7.9), i.e. $(\varepsilon_{sm} - \varepsilon_{cm})$ should be increased by multiplying it by $(h - x)/(d - x)$. This modified strain should then be used in conjunction with BS EN 1992-1-1:2004, Expression (7.14) [see Figure 6b)]

When the tension reinforcement is arranged in more than one layer, crack width calculations should be carried out using the outermost bars. In the calculation of $h_{c,ef}$, dimension d should be taken as the depth to the centre line of the bars in the outermost layer. Also the value of $h_{c,ef}$ should be limited to $(c + 0.5\phi) + 0.5$ (the spacing between the centre line of bars in adjacent layers) [Figure 6c)].

For sections in which the tension reinforcement is distributed through the depth of the section the procedure in **2.22** applies (Figure 7). In this method, the strain ε_m is the strain at the surface at the position where the crack width is sought.

2.21.3 Crack width due to restrained imposed deformation

BS EN 1992-1-1:2004 does not provide sufficient guidance on calculating crack width due to restrained imposed deformation but it is covered in BS EN 1992-3:2006. CIRIA C660 [19] deals with this topic more fully.

BS EN 1992-3:2006, Expression (M.3) may also be used for the calculation of crack widths caused by restrained imposed deformation generally using the restraint factors in BS EN 1992-3:2006. It should be noted that the restraint factors in BS EN 1992-3:2006 make an allowance for creep and no further allowance should be made. For suspended slabs the restraint factor may be assumed to be in the range 0,2 to 0,4.

Figure 6   **Typical cases of crack width calculations**

Key

1   $h_{c,ef}$ when < $(c + \phi/2)$

S bar spacing

a) $S \leq 5(c + 0.5\phi/2)$

Crack width calculated using strain at this level

Key

1   Neutral axis

2   $(\varepsilon_s - \varepsilon_m)$, calculated at this depth using BS EN 1992-1-1:2004, Expression (7.9)

3   Crack width calculated using strain at this level

S bar spacing

b) $S > 5(c + 0.5\phi/2)$

Key

1   Limiting value of $h_{c,eff} = c + 0.5(c + s_b)$

s_b Spacing between centre of bars in adjacent layers

c) Multiple layers of tension bars
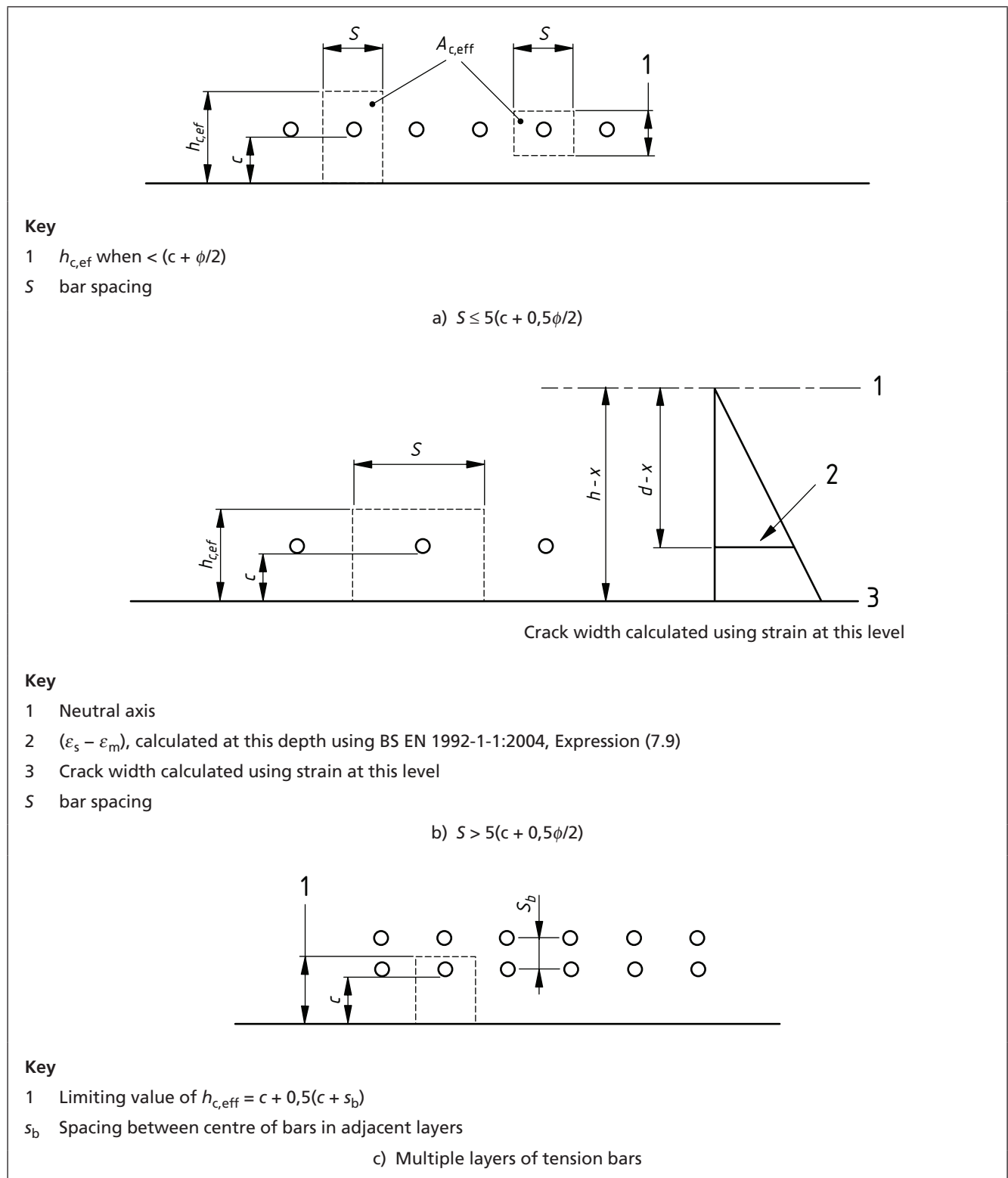
2.22   **Crack widths for non-rectangular tension zones and irregular bar layouts**

The crack width w_k calculation model in BS EN 1992-1-1:2004, **7.3.4** is related to rectangular tension zones. For other section shapes, the following method may be used to calculate crack width (see Figure 7).

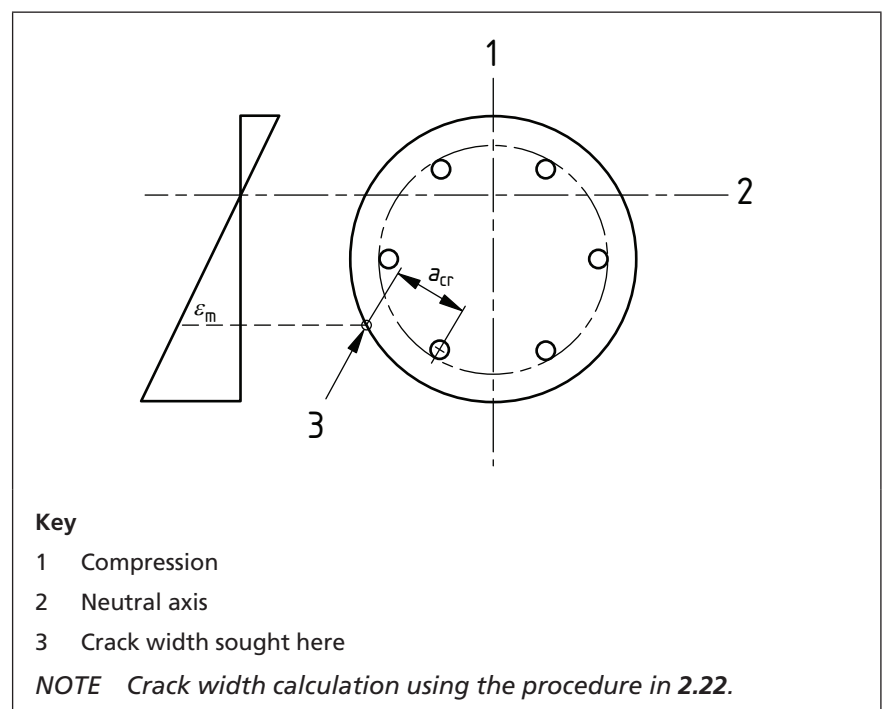$$w_k = \{3a_{cr}\,\varepsilon_m/[1+ 2(a_{cr} - c)/(h - x)]\}$$

where

a_{cr} is the distance from the point considered to the surface of the nearest longitudinal bar;

c, h and x are as defined in BS EN 1992-1-1:2004, **7.3**;

ε_m is the average strain at the level at which cracking is considered.

Figure 7   **Crack width calculations – non-rectangular tension zones**

Key

1   Compression

2   Neutral axis

3   Crack width sought here

*NOTE   Crack width calculation using the procedure in **2.22**.*

The strain ε_m should be calculated assuming:

a)   plane sections remain plane;

b)   concrete in compression is elastic (with the modulus of elasticity incorporating the appropriate allowance for creep);

c)   the reinforcement remains elastic; and

d)   the concrete stress varies linearly from zero at the neutral axis to 0,7 N/mm^2 at the extreme fibre in tension.

2.23   **Deflection control [BS EN 1992-1-1:2004, 7.4]**

2.23.1   **General considerations [BS EN 1992-1-1:2004, 7.4.1]**

BS EN 1990:2002+A1:2005 recommends that serviceability criteria should be specified for each project and agreed with the client. However, BS EN 1992-1-1:2004 sets out deflection limits and procedures. These are therefore likely to be basis of serviceability

design generally. If increased deflection limits are acceptable on a particular project, higher values of span/depth than those indicated in the code may be used; in such cases the design may not be considered to comply with BS EN 1992-1-1:2004. The designer should appreciate that there are many uncertainties that affect the reliability of deflection calculations. A factor that is of particular significance in the assessment of the deflection of slabs is the possible variation in tensile strength of concrete in the structure. This arises because the design load for deflections is commonly close to the load that causes cracking moment. It would therefore be prudent to consider a range of values rather than a single value of deflection.

2.23.2   Cases where calculations may be omitted [BS EN 1992-1-1:2004, 7.4.2]

2.23.2.1 For structures that remain propped during construction until the concrete attains the specified design strength, the deflections are likely to be within the limits given in BS EN 1992-1-1:2004 if the values used for span/depth ratios either conform to BS EN 1992-1-1:2004, Table 7.4N or BS EN 1992-1-1:2004, Expression (7.16a) and Expression (7.16b).

2.23.2.2 However, where the formwork is struck without back propping or the structure is loaded before the concrete attains the specified design strength, it is necessary to undertake detailed calculation for deflection. In these calculations appropriate early age properties of concrete should be used [20].

2.23.2.3 BS EN 1992-1-1:2004 allows the span/depth ratio obtained from Expression (7.16) to be modified by Expression (7.17), namely, $(310/\sigma_s) = (500/f_{yk})(A_{s,prov}/A_{s,req})$. The following should be noted.

a)   The value of σ_s to be used. The code states that it is the tensile steel stress at mid-span (or at support for cantilevers) under the design load at SLS. As verification is carried out under quasi-permanent loading, it might be thought that σ_s should be calculated under this loading. However, the background document produced by the project team [21] suggests that σ_s is the stress obtained under characteristic loading at serviceability limit state. This will also be more consistent with Expression (7.17). Therefore the following alternatives are recommended for the modification factor for the span/depth ratio obtained from Expressions (7.16a) or (7.16b):

   •   Factor = $(500/f_{yk})(A_{s,prov}/A_{s,req})$. The value of $(A_{s,prov}/A_{s,req})$ should not be taken as more than 1,5; or

   •   Factor = $(310/\sigma_s)$ in which σ_s is the stress calculated under characteristic combination of loads at serviceability limit state. The value of $(310/\sigma_s)$ should not be taken as more than 1,5.

b)   The reasons for imposing the upper limit of 1,5 include that:

   •   the increase in the reinforcement is insufficient to compensate for the reduction in the moment of inertia due to the reduction in member depth;

   •   the increase in $(310/\sigma_s)$ is greater than $(A_{s,prov}/A_{s,req})$; and

   •   deflections progressively exceed (span/250) as the ratio $(A_{s,prov}/A_{s,req})$ increases.

2.23.2.4 The method of determining the reinforcement ratio ρ requires clarification as it is not spelled out in BS EN 1992-1-1:2004. For rectangular sections, ρ is defined as $\rho = A_s/bd$. Excessive deflections can arise if the span/depth ratio for T sections is obtained using $\rho = A_s/b_{eff}d$ where b_{eff} is the effective flange width. Limited calibrations have shown that deflections would be closer to current practice, if ρ is defined at mid span for T sections as A_s divided by the area of concrete above the centroid of the tension steel.

2.23.3 Checking deflections by calculation [BS EN 1992-1-1:2004, 7.4.3]

Cracking has a significant effect on deflections. Although deflection is generally verified using quasi-permanent combination of loading, more severe loading causing significant cracking in the element in the past and in the future will lead to increased deflection. This may be allowed for by choosing a value for ζ in BS EN 1992-1-1:2004, Expression (7.18) corresponding generally to frequent combination of loading.

2.24 Column reinforcement [BS EN 1992-1-1:2004, 9.5]

2.24.1 Longitudinal reinforcement [BS EN 1992-1-1:2004, 9.5.2]

BS EN 1992-1-1:2004, **9.5.2** allows a national choice for the maximum amount of longitudinal reinforcement in columns.

BS EN 1992-1-1:2004, **9.5.2** recommends limits to the amount of longitudinal reinforcement, namely, 4% generally and 8% at laps. BS EN 1992-1-1:2004 permits higher values if it can be shown that the integrity of concrete is not affected. This may be deemed to have been achieved if there is adequate space to place and compact the concrete.

At laps, the width occupied by the reinforcement in any layer should not exceed 40% of the width of the section at that layer.

2.24.2 Transverse reinforcement [BS EN 1992-1-1:2004, 9.5.3(3)]

The guidance in BS EN 1992-1-1, **9.5.3**(3) should be followed. The more onerous recommendations for transverse reinforcement given in BS EN 1992-1-1:2004, **8.7.4.2** need not be applied to columns, although they should apply to bars permanently in compression.

National Amendment 1 to NA to BS EN 1992-1-1:2004 introduced more stringent requirements for transverse reinforcement in columns that use concrete class > C50/60. This was introduced to maintain similar ductility in columns over the whole range of concrete strengths. The model is based on BS EN 1998-1.

2.25 Tying systems [BS EN 1992-1-1:2004, 9.10]

Reinforcement used as ties should possess adequate ductility. BS EN 1992-1-1:2004, Class A reinforcement is not considered suitable for this purpose.

2.26   Tying requirements to comply with building regulations throughout the UK [BS EN 1992-1-1:2004, 9.10.2]

2.26.1   Vertical ties

BS EN 1992-1-1:2004, **9.10.2** requires that vertical ties are provided only in panel buildings of five storeys or more. However, the guidance documents to the building regulations (Approved Document A in England and Wales [22] and Domestic and Non-Domestic Technical Handbooks in Scotland [23,24]) recommends that such ties are provided in buildings that fall into Class 2B and 3 as defined in Section 5 of the Approved Document A to the Building Regulations 2000 [1] and Risk Groups 2B and 3, as defined in Section 1.2 of the Domestic and Non-Domestic Technical Handbooks to Building (Scotland) Regulations [2].

The details in the guidance documents are reproduced, as follows.

a)   "Each column and each wall carrying vertical load should be tied continuously from the lowest to the highest level. The tie should be capable of carrying a tensile force equal to the design load likely to be received by the column or wall from any one storey under accidental design situation [i.e. loading calculated using BS EN 1990:2002+A1:2005, Expression (6.11b)].

b)   "Where ties described in a) are not provided or where the layout incorporates key elements the failure of which would cause the collapse of more than a limited portion close to the element in question, the vertical member in question should be demonstrated for "non-removability". Non-removability may be assumed if the element and its connections are capable of withstanding a design load of 34 kN/m^2 at ultimate limit state applied from any direction to the projected area of the member together with the reaction from the attached components, which should also be assumed to be subject to a loading of 34 kN/m^2. The latter may be reduced to the maximum reaction that can be transmitted by the attached component and its connection.

c)   "In cases where it is impossible or inappropriate to provide effective vertical ties in accordance with a) in all or some of the vertical load bearing elements, at each storey in turn, each vertical load bearing element (other than a key element) should be considered removed one at a time and an alternative load path should be provided for the elements normally supported by the element that is removed."

2.26.2   Anchorage of precast floor and roof units and stair members

BS EN 1992-1-1:2004 does not cover anchorage of precast floor and roof units and stair members explicitly and the following recommendations should be followed.

a)   All precast floor, roof and stair members should be effectively anchored whether or not such members are used to provide other ties required in BS EN 1992-1-1:2004, **9.10.2**.

b)   The anchorage described in a) should be capable of carrying the dead weight of the member to that part of the structure that contains the ties.

These recommendations apply to precast units and stairs incorporated into concrete construction. Where they are incorporated into other forms of construction (e.g. masonry, structural steel or timber) the recommendations of the relevant Eurocode and its National Annex should be followed.

2.27 Modification of partial factors for materials [BS EN 1992-1-1:2004, Annex A]

BS EN 1992-1-1:2004, Annex A provides guidance on the modification of partial factors for materials. The conditions stipulated in the annex should be observed before advantage is taken of the reduction.

It should be noted that factor η noted in BS EN 1992-1-1:2004, **A.2.3** should not be applied to γ_c in conjunction with f_{ck} values derived from in-situ testing of structures in accordance with BS EN 13791 (and its complementary Standard BS 6089) as a factor of 0,85 has already been applied in the derivation of f_{ck}. The reduced γ_c may be applied directly to the measured in-situ strengths directly.

When partial factors are reduced for the ultimate limit state, appraisal of the serviceability behaviour should be undertaken. In general it will not be appropriate to use "deemed to satisfy" rules in BS EN 1992-1-1:2004 for deflection.

2.28 Detailing rules for particular situations [BS EN 1992-1-1:2004, Annex J]

2.28.1 General

NA to BS EN 1992-1-1:2004 declares that BS EN 1992-1-1:2004, Annex J is not applicable in the UK. An alternative version, which may be applied in the UK, is given in Annex B to this Published Document.

The main reasons for not recommending the use of BS EN 1992-1-1:2004, Annex J in the UK include the impracticality of using surface reinforcement and the method of defining "moderate" and "large" opening moments in a frame corner. These two issues are discussed in **2.28.2** and **2.28.3**.

2.28.2 Surface reinforcement

BS EN 1992-1-1:2004, Annex J recommends the use of surface reinforcement:

a)   for crack control when large diameter bars are used; and

b)   for preventing the falling off of concrete when the axis distance to the main reinforcement exceeds 70 mm.

Bearing in mind that NA to BS EN 1992-1-1:2004 defines a large diameter bar as any bar greater than 40 mm in diameter, the practical cases when surface reinforcement is likely to be needed are likely to be extremely limited. In such cases it is recommended that crack control is based on calculation rather than on the "deemed to satisfy" tables.

When the axis distance to main reinforcement exceeds 70 mm, it is recommended that consideration is given to other measures, such as the application of plaster, vermiculite or the provision of a false ceiling as a fire barrier.

2.28.3 Moderate and large opening moments

As high strength concrete is permissible in BS EN 1992-1-1:2004, "large" and "moderate" moments in a frame corner have been defined in Annex B using the mechanical reinforcement ratio $(A_s f_{yd})/(A_c f_{cd})$ rather than the geometric ratio used in BS EN 1992-1-1:2004, Annex J. This reflects the crushing/splitting strengths of the high strength concrete, which can sustain higher moments.

2.28.4 Appraisal and testing of structures and components during construction

The guidance in Annex C may be followed.

3 BS EN 1992-1-2:2004, Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design

The tabular methods in BS EN 1992-1-2:2004, **5.3** for assessing the fire resistance of columns are limited to braced structures. However, at the discretion of the designer, the methods given in BS EN 1992-1–2:2004 for columns may be used for the initial design of unbraced structures. In critical cases the chosen column sizes should be verified using BS EN 1992-1-2:2004, Annex B.

Adoption of these tabular methods for both braced and unbraced structures is likely to be acceptable because:

a) historically BS 8110 (Parts 1 and 2), which has been replaced by BS EN 1992-1-1:2004, did not make any distinction between braced and unbraced construction in fire design; and

b) the column sizes in BS EN 1992-1-2:2004 generally exceed those that were used in BS 8110 (Parts 1 and 2).

4 BS EN 1992-3:2006, Eurocode 2: Design of concrete structures – Part 3: Liquid retaining or containing structures

Annex D of this standard should be used instead of BS EN 1992-3:2006, Annex K as a source of information on the effect of temperature on the properties of concrete.

Annex A (informative)

Standards to used in conjunction with BS EN 1992-1 (all parts)

BS EN 1992-1 (all parts) is expected to be used generally in conjunction with European Standards (and their UK National Annexes) for loading, materials and execution. Some principal codes are as follows.

a) *Basis of structural design*: BS EN 1990:2002+A1:2005 and its UK National Annex. These define among other things the load combinations for different design situations and provide values for partial factors.

b) *Actions*: BS EN 1991 (all parts) and their UK National Annexes

c) *Concrete*: BS EN 206-1 and the complementary standards BS 8500-1:2006 and BS 8500-2:2006 are the British Standards for concrete.

d) *Reinforcing steel*: BS EN 10080:2005, in conjunction with BS 4449:2005, BS 4482:2005, BS 4483:2005 and BS 8666:2005.

e) *Prestressing steel*: until BS EN 10138 (all parts) [8] is published, BS 5896 may be used for prestressing steel.

f) *Execution and workmanship*: BS EN 13670:2009. The UK National Annex is appended to the UK adoption of the EN. National Structural Concrete Specification (4th edition) conforms to BS EN 13670:2009 and is useful when drafting a project specification.

g) In areas not specifically mentioned in a) to f), the designer may consider using current practice or current British Standards but the designer has to be satisfied that they are compatible with BS EN 1992-1 (both parts) and that the resulting reliability would be acceptable.

Annex B (informative)

Detailing rules for particular situations

B.1 General

The recommendations and guidance in this annex should be used as a replacement for BS EN 1992-1-1:2004, Annex J.

B.2 Surface reinforcement

In the UK the use of surface reinforcement is considered impractical. As an alternative it is recommended: that when:

a) large diameter (>40 mm) bars are used, calculations are undertaken to verify crack widths; and

b) when axis distance to main reinforcement is greater than 70 mm, surface protection such as plaster or vermiculite is used or a false ceiling is provided as a fire barrier.

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[8] In preparation.

B.3 Frame corner

B.3.1 General

The concrete strength $\sigma_{Rd,max}$ should be determined in accordance with BS EN 1992-1-1:2004, **6.5.2** (compression zones with or without transverse tension).

B.3.2 Frame corner with closing moments

B.3.2.1 For approximately equal depths of column and beam ($2/3 < h_2/h_1 < 3/2$) [see Figure B.1a)] no check of link reinforcement or anchorage lengths within the beam column joint is required, provided that all the tension reinforcement of the beam is bent around the corner.

B.3.2.2 Figure B.1b) shows a strut and tie model for $h_2/h_1 < 2/3$. Tanθ should lie between 0,4 and 1.

It should be verified that F_{cd3} and F_{td1} (shown in Figure B.1b) are less than the shear resistance $V_{Rd,max}$, when determined using BS EN 1992-1-1:2004, Expression (6.9) and taking z to be $0,8h_2$.

B.3.2.3 The anchorage length l_{bd} should be determined for the force $\Delta F_{td} = F_{td2} - F_{td1}$.

B.3.2.4 Reinforcement should be provided for transverse tensile forces perpendicular to an in-plane node.

B.3.3 Frame corner with opening moments

B.3.3.1 For approximately equal depths of column and beam the strut and tie models given in Figure B.2a) and Figure B.3a) may be used.

Reinforcement should be provided as a loop in the corner region or as two overlapping U bars in combination with inclined links as shown in Figure B.2b) and Figure B.3b)).

B.3.3.2 For large opening moments a diagonal bar and links to prevent splitting should be considered as shown in Figure B.3b).

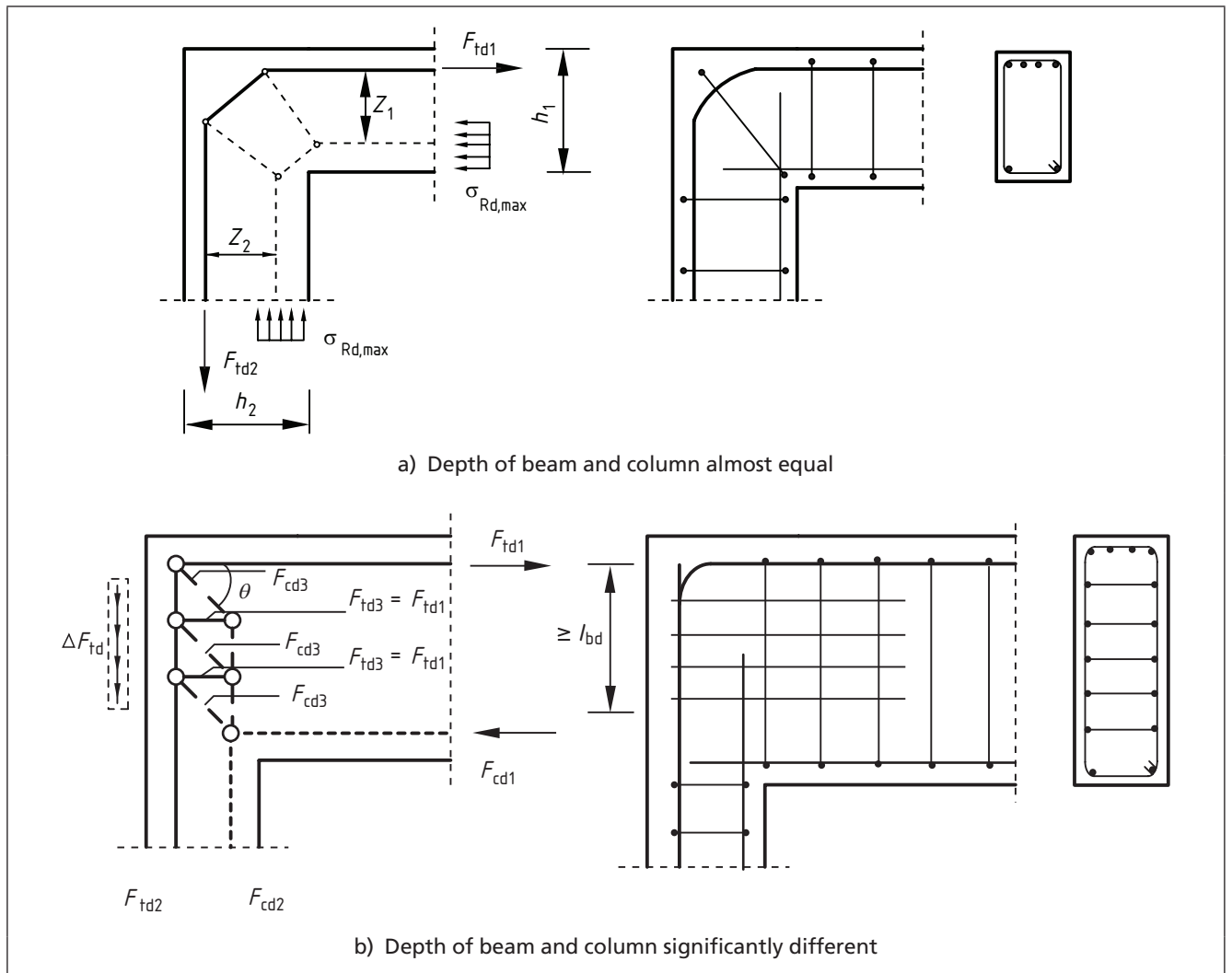Figure B.1 **Frame corner with closing moment – Model and reinforcement**

a) Depth of beam and column almost equal

b) Depth of beam and column significantly different

Figure B.2 **Frame corner with moderate opening moment ($A_s f_{yd}/A_c f_{ck}$) > 0,25**

a) Strut and tie model

b) Detailing of reinforcement
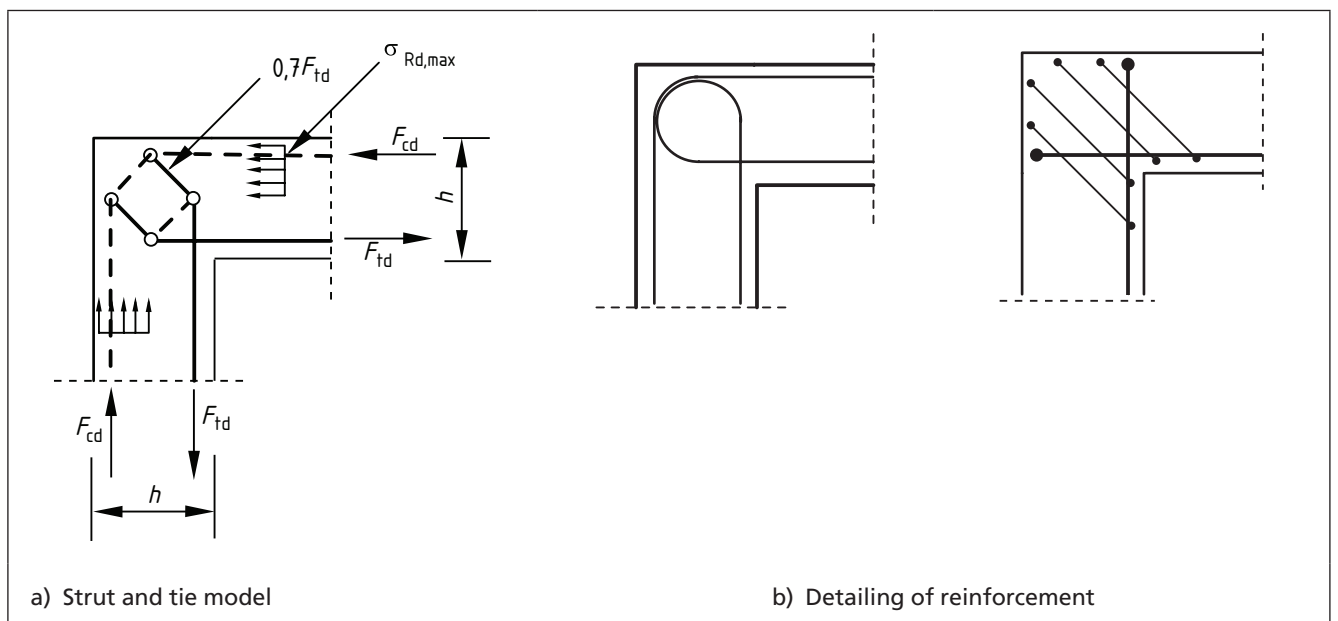
Figure B.3   **Frame corner with large opening moment ($A_s f_{yd}/A_c f_{ck}$) > 0,25**

a) Strut and tie model   b) Detailing of reinforcement
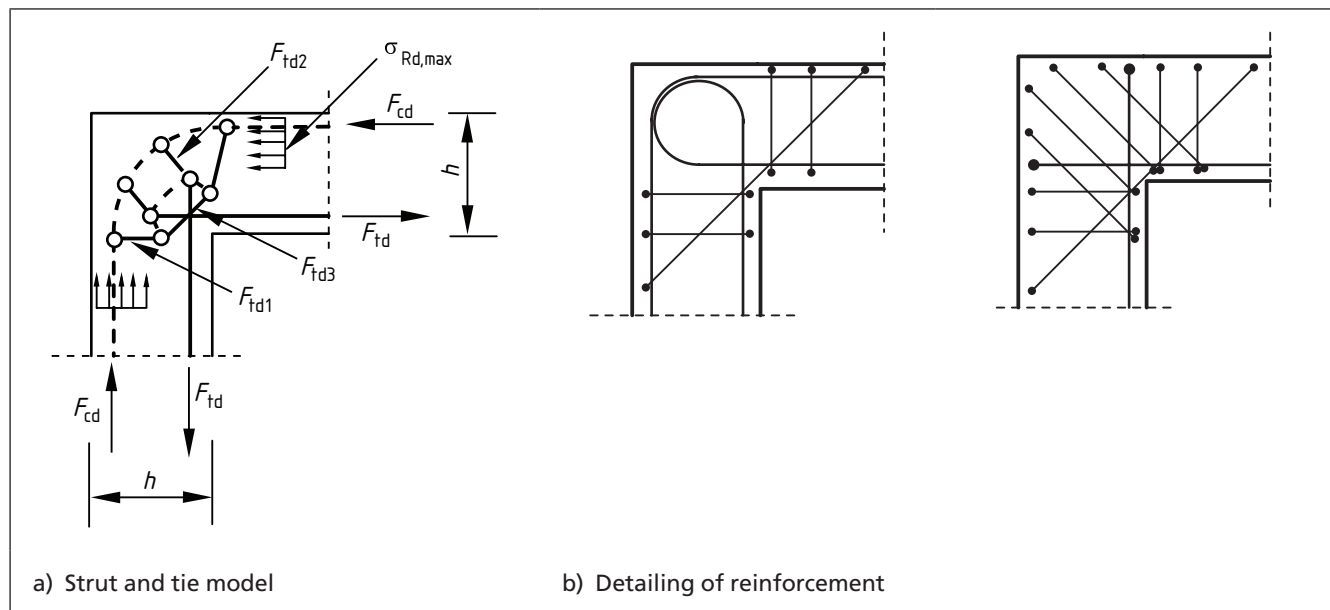
B.4   Corbels

B.4.1 Corbels ($a_c < z_0$) may be designed using strut-and-tie models as described in BS EN 1992-1-1:2004, **6.5** (see Figure B.4). The inclination of the strut is limited by $1{,}0 \leq \tan\theta \leq 2{,}5$.
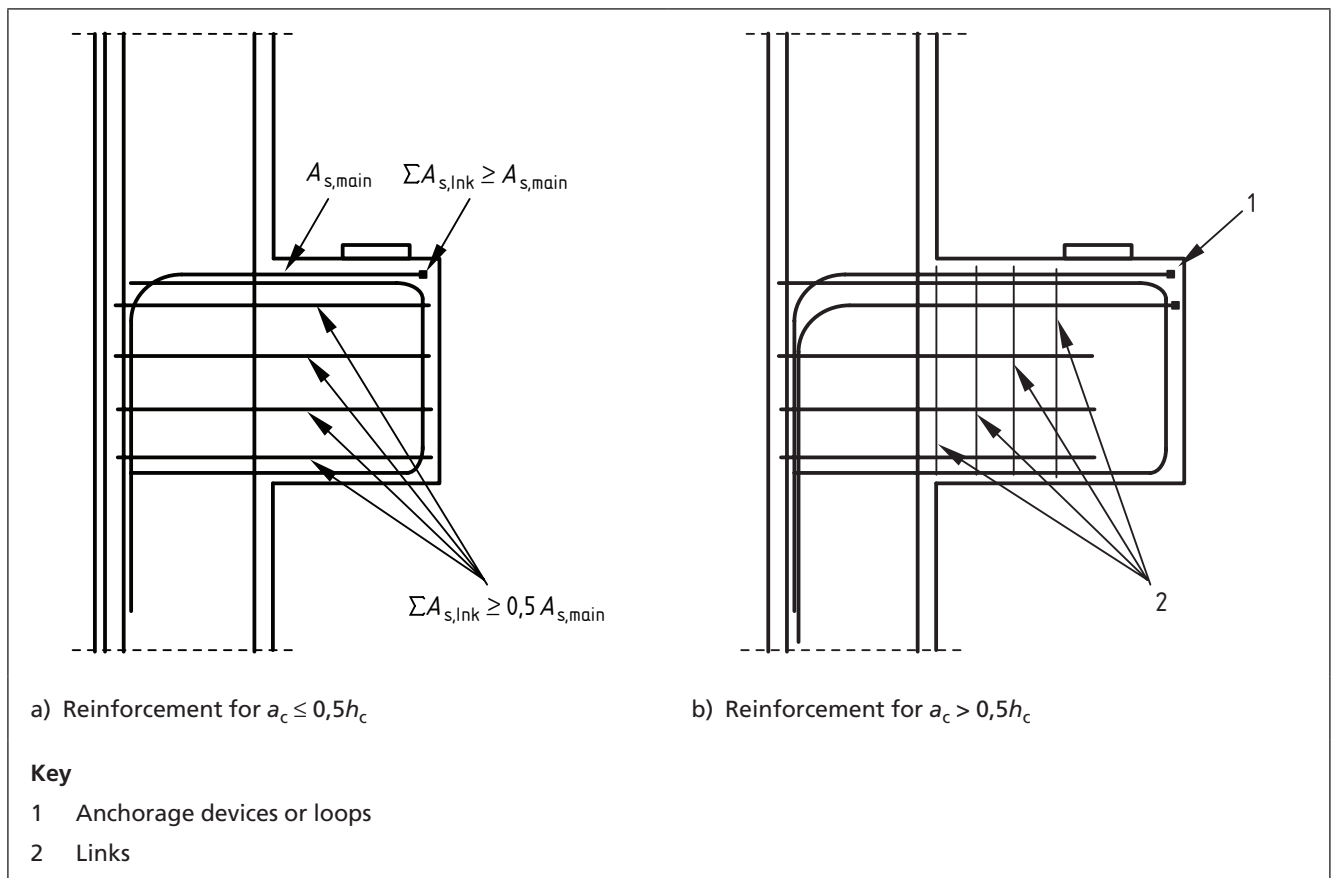
Figure B.4   **Corbel strut-and-tie model**
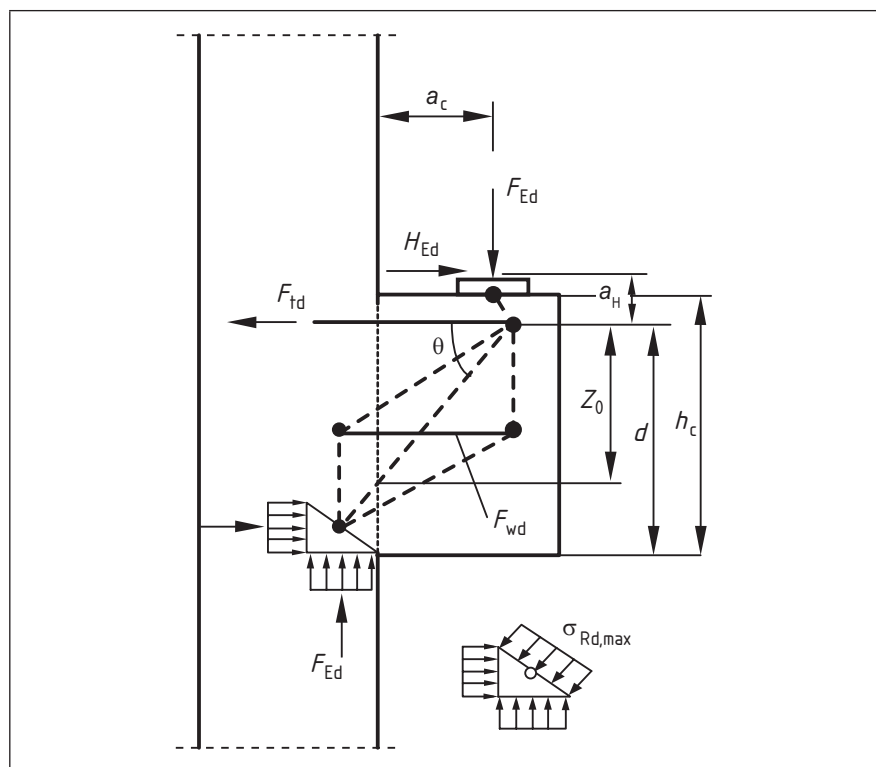
B.4.2 If $a_c < 0{,}5h_c$ closed horizontal or inclined links with $A_{s,lnk} \geq 0{,}5A_{s,main}$ should be provided in addition to the main tension reinforcement [see Figure B.5a)].

B.4.3 If $a_c > 0,5h_c$ and $F_{Ed} > V_{Rd,c}$ (see BS EN 1992-1-1:2004, **6.2.2**), closed vertical links $A_{s,lnk} \geq 0,5F_{Ed}/f_{yd}$ should be provided in addition to the main tension reinforcement [see Figure B.5b)].

B.4.4 The main tension reinforcement should be anchored at both ends. It should be anchored in the supporting element on the far face and the anchorage length should be measured from the location of the vertical reinforcement in the near face. The reinforcement should be anchored in the corbel and the anchorage length should be measured from the inner face of the loading plate.

B.4.5 If there are special requirements for crack limitation, inclined stirrups at the re-entrant opening can be effective.

Figure B.5 **Corbel detailing**

a) Reinforcement for $a_c \leq 0,5h_c$

b) Reinforcement for $a_c > 0,5h_c$

Key

1   Anchorage devices or loops

2   Links

Annex C (informative) # Appraisal and testing of structures

C.1   General

This annex refers to methods for appraisal and, where necessary, for testing whole structures, finished parts of a structure or structural components during the construction phase. It is assumed that the structure and components have been designed in accordance with BS EN 1992-1-1:2004, BS EN 1992-1-2:2004 and their respective National Annexes. The guidance may be followed for existing structures provided their performance is calibrated against a design to the above reference standards.

The following provides general guidelines only.

C.2   **Purpose of testing**

The methods noted below may be considered appropriate in any of the following circumstances:

a)   materials used in construction are suspected to be defective or sub-standard;

b)   construction is considered to be outside the design and specification;

c)   where there are visible defects;

d)   where a check is required on the quality of the construction or manufacture of precast units.

C.3   **Basis of approach**

The basic objective of appraisal under the circumstances described in **C.2** is to assess the structure as built and to decide whether or not it meets the requirements of the original design.

The type and extent of any tests used in a particular case should be chosen to achieve this objective and should be agreed in advance by all the parties concerned, both in principle and in detail. The tests should be relevant. Based on the information so obtained and on an examination of all other relevant factors, a judgement can be made on the acceptability of the structure.

In general these procedures should be systematic and incremental, i.e. the methods given in **C.4** should be used first and only if there is still doubt should those in **C.6** be considered.

C.4   **Check tests on structural concrete**

C.4.1   **General**

This subclause covers tests used to determine the quality of the materials used in the structure as built; the values of the material parameters so obtained may then be used in calculations to appraise the structure. The prime concern is with the measurement of strength in situ, either directly or indirectly, but tests may also be required to determine concrete cover and integrity, material composition, the presence of defects or contaminants, etc. Available test techniques are listed in [25] together with an assessment of their applicability, advantages and limitations.

C.4.2   **Strength of concrete in the structure**

Test methods to be used for the determination of the in-situ strength of concrete and the methods of evaluation of the results are given in BS EN 13791 and the complementary standard BS 6089 (under preparation). Actual testing and reporting results are covered in BS EN 12504, Parts 1 to 4 and their UK National Annexes, where published.

C.5 Load tests of structures or parts of structures

C.5.1 General

A load test may be deemed necessary to check on either strength or serviceability. It should be recognized that loading a structure to its ultimate design loadings might impair its subsequent performance in service, without necessarily giving a true measure of load carrying capacity. While such overload tests may sometimes be justified (**C.6**), it is generally recommended that the structure be loaded to a level appropriate to serviceability limit states. If sufficient measurements of deformations are taken, then these, together with the results from the test described in **C.5.2** can be used to calibrate the original design in predicting the ultimate strength and long term performance of the structure.

Detailed recommendations on test procedures are given in [25] with background information being provided in [26] and [27]. Some general principles are given in **C.5.2** to **C.5.4**.

C.5.2 Test loads

The total load to be carried W should not be less than 1,0 times the characteristic permanent action plus 1,0 times the characteristic variable action, and should normally be the greater of:

a) the sum of the characteristic permanent action and 1,25 times the characteristic variable action; or

b) 1,125 times the sum of the characteristic permanent action and characteristic variable action.

In deciding on a suitable figure for this and how to apply the test load on the structure, due allowance should be made for finishes, partitions, etc., and for any load sharing that could occur in the completed structure, i.e. the level of loading should be representative and capable of reproducing the proper internal force system reasonably closely.

The test load should be applied and removed incrementally, while observing all proper safety precautions. The test loading should be applied at least twice, with a minimum of 1 hour between tests, and allowing 5 min after a load increment is applied before recording deformation measurements. Consideration may also be given to a third application of load, which is left in position for 24 hours.

C.5.3 Assessment of results

In determining deformation measurements, due allowance should be made for changes in environmental conditions that have occurred during the test.

The main objective in assessing the results is to compare the measured performance with that expected on the basis of design calculations. This will require due allowances to be made for any differences in material strength or stress or other characteristic in the as-built structure, compared with that assumed in design. These material properties should be determined as accurately as possible.

C.5.4   **Test criteria**

In assessing test data and recalculation procedures, the following should be considered:

a) the initial deflection and cracking should be in accordance with the design requirements;

b) where significant deflections have occurred under the normal loads given in **C.5.2**, the percentage recovery after the second loading should be at least equal to that for the first loading cycle, and should be at least 75% for reinforced concrete and 85% for prestressed concrete, in which design tensile stresses (under characteristic loading combination) do not exceed the flexural tensile strength in pretensioned members and 0,8 of the flexural tensile strength in post tensioned members;

c) the structure should be examined for unexpected defects, which should then be evaluated in the recalculation procedures.

NOTE   *Where the measured deflections are very small (e.g. span/1 000), estimates of recovery become meaningless.*

C.6   **Load tests on individual precast units**

Load tests on individual precast units might be necessary for reasons in **C.2**a) to c). In these cases the procedures should be in accordance with **C.3**, **C.4** and **C.5**.

If load testing is also required as a check on the quality of the units for the acceptance of new units, the procedure may again be in accordance with **C.3**, **C.4** and **C.5** or as determined by the technical schedule in a satisfactory quality assurance system. Sampling rates should be as given in the technical schedule or as in the specification. The basis for the overall approach should be as noted in **C.2** for the assessment of both serviceability and strength, in which case overload tests will not normally be required. Should doubt exist about the ultimate strength of a series of units, then tests to failure might be necessary, at a rate to be agreed by parties concerned. In such tests the performance should be in accordance with that expected from the design calculations. In general, the ultimate strength should exceed the ultimate load by a margin of at least 5%; moreover, the deflection up to the design ultimate load, should not exceed span/40.

Annex D (informative)   # Effect of temperature on the properties of concrete

D.1   **General**

The recommendations and guidance in this annex should be used as a replacement for BS EN 1992-3:2006, Annex K.

This Annex covers the effects on the material properties of concrete of temperatures in the range –25 °C to +200 °C. Properties covered are: strength and stiffness, creep and transitional thermal strain.

In all cases the changes in properties are strongly dependant on the particular type of concrete used and this annex should not be considered to provide more than general guidance.

D.2 Material properties at sub-zero temperatures

When concrete is cooled to below zero, its strength and stiffness increase. This increase depends mainly on the moisture content of the concrete: the higher the moisture content, the greater is the increase in strength and stiffness. Such increases should be determined for the specific concrete and moisture contents expected. It should be noted that the enhancement in properties would apply only to concrete that would be permanently below −25 °C.

Creep at sub-zero temperatures may be taken to be 60% to 80% of the creep at normal temperatures. Below −20 °C creep may be assumed to be negligible.

D.3 Material properties at elevated temperatures

Information on the compressive strength and tensile strength of concrete at temperatures above normal may be obtained from BS EN 1992-1-2:2004, **3.2.2**.

The modulus of elasticity of concrete may be assumed to be unaffected by temperature up to 50 °C. For higher temperatures, a linear reduction in modulus of elasticity may be assumed up to a reduction of 20% at a temperature of 200 °C.

For concrete heated prior to loading, the creep coefficient may be assumed to increase with increase in temperature above normal (assumed as 20 °C) by the appropriate factor from Table D.1.

Table D.1 **Creep coefficient multipliers to take account of temperature where the concrete is heated prior to loading**

Temperature °C	Creep coefficient multiplier
20	1,00
50	1,35
100	1,96
150	2,58
200	3,20

NOTE   The values in the table have been deduced from CEB Bulletin 208 [28] and are in good agreement with multipliers calculated on the basis of an activation energy for creep of 8 kJ/mol.

In cases where the load is present during the heating of the concrete, deformations will occur in excess of those calculated using the creep coefficient multipliers given in Table D.1. This excess deformation, the transitional thermal strain, is an irrecoverable, time-independent strain which occurs in concrete heated while in a stressed condition. The

maximum transitional thermal strain may be calculated approximately from the expression:

$$\varepsilon_{Tr} = \kappa \sigma_c \varepsilon_{Th} / f_{cm} \tag{D.1}$$

where

    κ is a constant obtained from tests and takes a value within the range $1{,}8 \leq \kappa \leq 2{,}35$;

    f_{cm} is the mean compressive strength of the concrete;

    ε_{Tr} is the transitional thermal strain;

    ε_{Th} is the free thermal strain in the concrete (= temperature change × the coefficient of expansion);

    σ_c is the applied compressive stress.

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