A cement and concrete industry publication

Properties of Concrete for use in Eurocode 2

How to optimise the engineering properties of concrete in design to Eurocode 2

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eurocode 2
QPA Providing Essential Materials
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Properties of concrete for use in Eurocode 2

Contents

Symbols

c	cover to reinforcement
c_p	specific heat
c_v	coefficient of variation
D	thermal diffusivity
E_c	tangent modulus
E_{cd}	design value of modulus of elasticity of concrete
$E_{c,eff}$	effective modulus of elasticity of concrete
E_{cm}	mean secant modulus of elasticity of concrete
f_{bd}	ultimate (design) bond stress
f_{cd}	design compressive strength
$f_{cd,fat}$	design fatigue strength
f_{ck}	specified characteristic cylinder compressive strength
$f_{ck,c}$	confined characteristic compressive strength
$f_{ck,cube}$	specified characteristic cube compressive strength
f_{cm}	mean concrete cylinder compressive strength
$f_{cm,cube}$	mean concrete cube compressive strength
f_{ctd}	design tensile strength
f_{ctk}	characteristic axial tensile strength of concrete
f_{ctm}	mean axial tensile strength
$f_{ctm,sp}$	mean splitting tensile strength
$f_{ctm,fl}$	mean flexural tensile strength
$f_{ct,sp}$	tensile splitting strength
f_{cu}	specified characteristic cube compressive strength (BS 8110 term)
s	coefficient for cement type used with the age function
$s_{r,max}$	crack spacing
t	time
α	coefficient applied to age function
α_c	coefficient of thermal expansion
α_{cc}	coefficient for long-term and loading effects on compressive strength
α_{ct}	coefficient for long-term and loading effects on tensile strength
$\beta_{cc}(t)$	age function for strength
γ_c	partial safety factor for strength of concrete
γ_{cE}	partial safety factor for strength of concrete used with E_{cm}
γ_m	partial safety factor for strength of a material
$\varepsilon_{ca}(t)$	autogenous shrinkage strain up to time t
$\varepsilon_{ca}(\infty)$	autogenous shrinkage strain at time $t = \infty$
$\varepsilon_{cc}(\infty,t_0)$	creep deformation at time $t = \infty$
ε_{cd}	drying shrinkage strain
ε_{cs}	total shrinkage strain
ε_{ctu}	tensile strain capacity
η_1	coefficient related to bond condition
η_2	coefficient related to bar diameter

λ_c	thermal conductivity
ρ	density
$\rho_{p,eff}$	ratio of area of reinforcement to effective area of concrete
ϕ	bar diameter
$\varphi (\infty, t_0)$	creep coefficient at time $t = \infty$
σ_c	constant compressive stress applied at time $t = t_0$

1. Introduction

In the design of concrete structures, engineers have the flexibility to specify particular concrete type(s) aimed at meeting the specific performance requirements for their project. For instance where calculated deflections exceed serviceability limits, the designer can achieve lower deflections by increasing the class of concrete and the associated modulus of elasticity, rather than by resizing members.

With this flexibility goes the responsibility for ensuring that the quality control in concrete production and subsequent site operations will enable the concrete as cast to meet the specified requirements in service.

Typically concrete is specified by compressive strength class, which indicates the characteristic compressive strength required. However, in design, a range of properties of concrete are used that are not normally part of the concrete specification. These may relate to both structural integrity and serviceability. BS EN 1992-1-1, (*Eurocode 2: Design of concrete structures,* Part 1-1 – *General rules and rules for buildings)* Section 3: *Materials* details these properties which are generally assumed to be related to the cylinder compressive strength, expressed either as the characteristic or the mean value, and are calculated using expressions which include one or other of these values.

1.1 Scope

This publication covers the background to the use of concrete properties in design, and is structured to provide guidance on:

- the range of concrete properties required in the design process.
- how each property is determined in BS EN 1992-1-1.
- how the property can be measured.
- how the measured value may be used in design.
- options for modifying the value of the property.

The guidance is intended to provide design engineers with a greater knowledge of concrete behaviour, so that they can optimise the use of the material aspects of concrete in their design.

Section 3 of BS EN 1992-1-1 gives principles and rules for normal- and high-strength concrete (15–105MPa cube strength) and for normal-weight concrete. Lightweight aggregate concrete (< 2200kg/m^3) is covered in section 11 of the Code and is not covered in this publication.

Guidance is given on the use of Eurocode2 (EC2) and on the corresponding UK National Annex (generally to Eurocode 2-1-1). Where a 'nationally determined parameter' which specifically applies to the UK is given, this is stated or denoted (NDP), and this value may be different for other CEN member countries.

EC2

Where an equation from Eurocode 2 is quoted, the Eurocode equation reference is highlighted alongside the equation in the text.

A list of European, national and international standards referred to in this publication is given under references at the back.

1.1.1 Mechanical properties

BS EN 1992-1-1 (Eurocode 2: *Design of concrete structures,* Part 1-1) sets out rules for the design of concrete structures and in table 3.1 gives recommended values for various mechanical properties of concrete for use in design. These property values are based on a number of assumptions and in general will be conservative. In most cases, these design values will be appropriate; however, in some circumstances the assumed design value may limit the design possibilities. Engineers who wish to take advantage of the full potential of concrete construction may therefore wish to look at the design values more closely to identify where changes may be cost-effective. This may be the case with the current trend to use higher-strength concrete, when serviceability considerations may start to control the design process.[1] As an example, if a higher value of modulus could be achieved, slab spans could be increased without increasing thickness. Use of high-strength concrete can also lead to lower shrinkage and creep values.

Designers may therefore wish to specify a value higher than the value from table 3.1 for a particular property and this guide provides information on how this may be achieved. The designer should, however, seek assurance from the contractor or specialist subcontractor that the concrete required to achieve the specified values can be supplied in practice – see Section 1.2.

In addition to compressive strength, the following mechanical properties of concrete are used in some design procedures, and guidance is provided in this publication on how targeted values may be achieved for normal-weight concrete:

- tensile and flexural strength
- bond strength
- modulus of elasticity
- tensile strain capacity
- creep.

Table 3.1 of BS EN 1992-1-1 provides values for the principal strength and deformation characteristics of concrete for a range of strength classes and this is replicated in Appendix A, Table A1.

1.1.2 Other properties

In addition to properties relating to strength and stiffness, a range of other properties may be required for design. Such properties dealt with in this publication include:

- autogenous shrinkage
- drying shrinkage
- coefficient of thermal expansion
- thermal conductivity
- specific heat
- fire resistance
- adiabatic temperature rise
- durability.

The achievement of ductility in a structure[2] is not covered in this publication. In the analysis of concrete structures, the formation of plastic hinges is based on the assumption that the reinforcement will continue to take the load while the reinforcement yields. BS EN 1992-1-1, cl 3.2.4 gives provisions for using reinforcement with different ductility. The use of fibres will also improve the ductility of concrete, but this is outside the scope of this publication and BS EN 1992-1-1.

1.2 Practical aspects of supply

Where the specifier wishes to establish if a particular value for a property is feasible for use in design, he should first consult with the concrete supplier who may have historic data available. However, it may be necessary to request an initial testing programme (prior to supply) where the relationship between this property and mix proportions and compressive strength can be established. Such testing can take some time and this must be adequately timetabled.

If the property values from the test programme have significant scatter, the specifier should allow for a degree of uncertainty by building in a margin for design purposes in the conversion from the property values to an equivalent compressive strength. The concrete specification should then either be based on the compressive strength class, and if appropriate the types of materials that are expected to provide the required performance; or alternatively it should be agreed with the producer that a particular concrete will satisfy the required property.

Most of the test methods for other properties listed in Section 1.1.1 and 1.1.2 will have a higher within-test coefficient of variation than for compressive strength and for this reason initial testing should be designed to establish the property relationship with compressive strength only, and compressive strength should remain the conformity test for concrete supply based on this relationship.

In circumstances in which specified properties may require concrete that is outside the normal range of production, it is advisable for the specifier to enter into early dialogue with the concrete producer. In particular, the following points should be noted:

- Additional lead time may be required for the procurement of materials and mix development and testing.
- Practical issues may need to be accommodated in concrete production and delivery.
- Specific contractual requirements may arise, in relation to procurement.
- Additional performance testing may be needed and the limitations on any non-standard methods should be understood.

2. Assumptions underlying Eurocode 2

Importantly, Eurocode 2 assumes that design and construction will:

- be subject to adequate supervision and quality control procedures.
- be carried out by personnel having the appropriate skills and experience.
- use materials and products as specified.
- meet the requirements for execution and workmanship given in ENV 13670 (due late 2008), *Execution of concrete structures,* and it's corresponding UK annex.

It is also assumed that the structure will be used in accordance with the design brief and be adequately maintained.

In addition, BS EN 1990, *Basis of structural design*, implies that design should be undertaken using limit state principles. Limit states are states beyond which the structure no longer fulfils the design intent.

- **Ultimate Limit States (ULS)** are associated with collapse or other forms of structural failure, for example, through flexural failure, shear failure, buckling, failure of anchorages.
- **Serviceability Limit States (SLS)** correspond to conditions beyond which specified service requirements are no longer met, for example, excessive deformation, excessive cracking or stress.

In design, both limit states are checked (or verified) as part of the design process for all relevant design situations. ULS calculations always use characteristic values and SLS calculations almost always use mean values.

3. Compressive strength

The only engineering property of concrete that is routinely specified is the characteristic compressive strength. This has a relationship to most other mechanical properties and provides the basis for estimating these.

It is important that the design strength of a structure, which is determined from either durability, fire design or structural design requirements, is established at the preliminary design stage. This will avoid having to recheck and/or amend a completed design as a consequence of an increased strength requirement to meet durability requirements for example, from which there could be implications. As an example, an increase of tensile strength as a result of going to a higher class of concrete, will mean the minimum steel ratio will need to be increased for crack control purposes.

3.1 Strength class

In BS EN 206-1: *Concrete – Specification, performance, production and conformity*, compressive strength is expressed as a strength class. BS EN 1992-1-1 uses the characteristic compressive cylinder strength f_{ck} (based on 2:1 cylinders) as the basis of design calculations. It also provides the basis for expressions in BS EN 1992-1-1 used to derive other concrete properties (for example, tensile strength, *E*-value, creep and shrinkage) although more precise values may be derived when necessary from testing in accordance with the relevant test standard.

While the specified 28-day characteristic strength is the most common input to the design, there are situations where it may be appropriate to use a higher strength for design. Such an instance includes where the structure will not be loaded for a long period after casting and the concrete is of a type and in a situation where its in-situ strength will continue to develop significantly beyond 28 days.

In addition, it may be necessary to know the strength at an early age, for example, for transfer of pre-stress, or for removal of props.

In the UK the compressive strength is tested using cubes (100mm or 150mm) rather than cylinders. A higher strength is obtained for cubes because the test machine platens offer greater lateral restraint due to the lower aspect ratio. In BS EN 206-1 the 2:1 cylinder strength is taken to be about 20% less than the cube strength for normal structural concrete but with higher strength classes, the cylinder strength achieves a higher proportion of the cube strength. To accommodate these differences, the strength class is defined by both the cylinder and the cube strength (for example, C30/37 C cube/cyl).

3.2 Characteristic strength, f_{ck}

The characteristic strength is that strength below which 5% of results may be expected to fall. Individual results below f_{ck} may be obtained but, in general, only need to be investigated if they fall more than 4MPa below f_{ck} (BS EN 206-1, cl 8.2, table 14).

3.3 Design strength

EC2 3.15

The design compressive strength of concrete, f_{cd}, according to BS EN 1992-1-1 is taken as:

$$f_{cd} = \alpha_{cc}\, f_{ck}/\gamma_c \qquad (1)$$

where

f_{ck} = characteristic cylinder compressive strength of concrete at 28 days

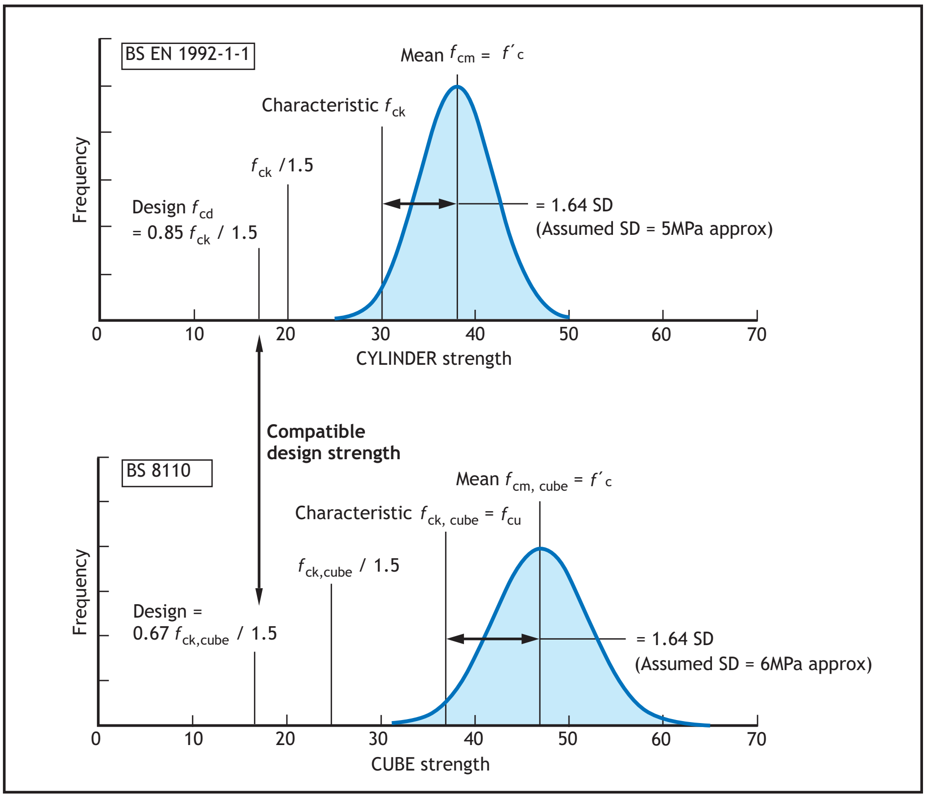
γ_c = partial (safety) factor for concrete

α_{cc} = a coefficient taking account of long-term effects on the compressive strength (which is reduced under sustained load) and unfavourable effects resulting from the way the load is applied.

Expression (1) is equivalent to the term $f_{cd} = 0.67 f_{cu}/\gamma_m$ used in BS 8110 (where f_{cu} is now represented as $f_{ck,cube}$). In each case the material safety factor (γ_c or γ_m) is 1.5. BS EN 1992-1-1 recommends that $\alpha_{cc} = 1$.

However, α_{cc} is an NDP and the UK National Annex to BS EN 1992-1-1 recommends that α_{cc} should be 0.85 for compression in flexure and axial loading and 1 for other phenomena (for example, shear, torsion and web compression – see PD 6687 Clause 2.3). It may also be taken conservatively as 0.85 for all phenomena. This leads to a design strength that is consistent with that of BS 8110 as shown in Figure 1 for strength class C30/37.

Figure 1
Compressive strength definitions to BS EN 1992-1-1 and BS 8110 for strength class C30/37.



3.4 Confined concrete

Confinement of concrete results in a modification of the effective stress–strain relationship. Confinement can be generated by links or cross-ties adequately anchored to resist bursting stresses. This results in an increased effective compressive strength, $f_{ck,c}$ and higher critical strains as outlined in BS EN 1992-1-1, Clause 3.1.9. The value of $f_{ck,c}$ is calculated using the expressions:

EC2 3.24

$$f_{ck,c} = f_{ck}\,(1000 + 5.0\,\sigma_2/f_{ck}) \qquad \text{for } \sigma_2 \le 0.05 f_{ck} \qquad (2)$$

EC2 3.25

$$f_{ck,c} = f_{ck}\,(1125 + 2.5\,\sigma_2/f_{ck}) \qquad \text{for } \sigma_2 > 0.05 f_{ck} \qquad (3)$$

where

σ_2 is the effective lateral stress due to confinement.

3.5 Target mean strength

Mechanical properties are used to check serviceability limit states and values are almost always related to the mean compressive strength and not the characteristic strength. For simplicity, the mean strength is assumed to be the characteristic strength plus 8MPa (cylinder), equivalent to plus 10MPa in terms of cube strength. Given the approximate nature of the relationships between the mechanical properties and the mean compressive strength, the use of a margin of 8MPa (cylinder) and 10MPa (cube) is usually adequate and there is no justification for using a lower margin.

The target mean strength, f_{cm}, is also the value used to establish the mix design and is intended to take account of the normal variability that will occur in concrete production. This margin of 8MPa for cylinders is consistent with a normal distribution with a standard deviation (SD) of about 5MPa:

$$f_{ck} = f_{cm} - 1.64\text{SD, where } 1.64\text{SD} = 8$$

Therefore

$\text{SD} = 8/1.64 \approx 5\text{MPa}$

The margin is 10MPa for cubes, which is equivalent to a standard deviation of about 6MPa. This is well within the capability of concrete produced from a certified plant. Target mean values for each strength class are shown in Table 1.

Table 1
Mean compressive cylinder and cube strength for different strength classes.

Mix designation	C12/16	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75	C70/85	C80/95	C90/105
Characteristic cylinder strength f_{ck}	12	16	20	25	30	35	40	45	50	55	60	70	80	90
Target mean cylinder strength f_{cm}	20	24	28	33	38	43	48	53	58	63	68	78	88	98
Characteristic cube strength $f_{ck,cube}$	16	20	25	30	37	45	50	55	60	67	75	85	95	105
Target mean cube strength $f_{cm,cube}$	26	30	35	40	47	55	60	65	70	77	85	95	105	115

3.6 Development of compressive strength with time

Numerous types of cement are available and in general, and unless specifically stated, it is assumed that the cement type will not affect the 28-day design properties of the concrete. However, the cement type has a significant effect on the rate of development of strength and other properties, and the concrete supplier should be able to provide historic strength development data. Alternatively BS EN 1992-1-1 expressions for calculating strength gain are given below. Appendix A, Table A2 provides details of the composition for a range of cements and combinations.

While design is usually based on the 28-day strength, BS EN 1992-1-1, sub-clause 3.1.2(6) gives an expression for the development of the mean compressive strength of concrete with time at 20°C as follows:

EC2 3.1

$$f_{cm}(t) = [\beta_{cc}(t)]\, f_{cm} \qquad (4)$$

where

$f_{cm}(t)$ is the mean compressive strength at age t days.

EC2 3.2

$$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\frac{28}{t}\right)^{0.5}\right]\right\} \qquad (5)$$

where

s is a coefficient which depends on cement type

= 0.20 for cement of strength classes CEM 42.5R, CEM 52.5N and CEM 52.5R (Class R)

= 0.25 for cement of strength classes CEM 32.5R, CEM 42.5N (Class N)

= 0.38 for cement of strength classes CEM 32.5N (Class S)

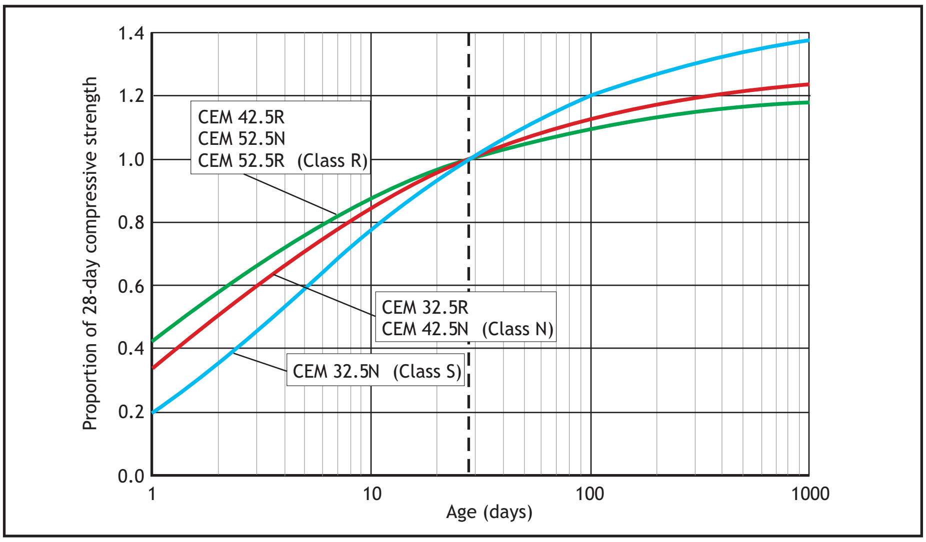
(where Class R = high early strength; Class N = normal early strength; Class S = slow early strength).

Usually the cement class will not be known at the design stage; however, generally class R should be assumed unless the following alternatives apply:

- Where ground granulated blastfurnace slag (ggbs) exceeds 35% of the cement combination or where fly ash (fa) exceeds 20%, class N may be assumed.
- Where ggbs exceeds 65% or fa exceeds 35%, Class S may be assumed.

Compressive strengths obtained from Expression (4) are shown in Figure 2. It should be noted that strength gain after 28 days is more dependent upon the cement type than the cement strength class. For example, the percentage strength gain after 28 days of a CEM I 42.5N concrete will be significantly lower than that for concrete made with, for example, CEM IIB-V 32.5 or CEM IIIA 32.5 cements, provided there is water for continued hydration.

Figure 2
Rate of compressive strength development at 20°C for different cement strength classes.



In reality there is a wide range of strength development dependant on a number of factors. If the designer has information that shows that the concrete to be supplied will gain strength more rapidly, this information could be used, for example, in serviceability calculations. BS EN 1992-1-1 notes that the estimated strength development beyond 28 days should not be used retrospectively to justify a non-conforming reference strength.

3.7 Strength in the structure

The strength obtained using standard test specimens will in the long term be greater than the actual compressive strength in the structure. This is due to a combination of factors including the process of manufacture and curing which is achieved more effectively in small test specimens. BS EN 13791 *Assessment of in-situ compressive strength in structures and pre-cast concrete components* requires that the minimum in-situ strength should be 0.85 times the strength of standard specimens. This factor is part of the material safety factor γ_m and should not be confused with α_{cc} which has the same magnitude.

The rate of strength development in the structure itself[3] will depend upon:

- type of concrete (mainly cement type and content)
- concrete placing temperature
- ambient temperature
- section thickness
- type of formwork
- curing temperature, for example, for precast elements.

A study by The Concrete Society to measure in-situ strength[4] and to assess the relationship between core strength and cube strength in a variety of elements indicated that the factor of 0.85 may not always be applicable. In elements using CEM I (Portland cement) subjected to high early-age peak temperature (in excess of about 60°C), the in-situ strength at 28 days (measured using 1:1 cores) achieved a value that was only about 65% of the cube strength. However, this was still accommodated within the material safety factor γ_c of 1.5 and continued strength development resulted in the in-situ strength achieving 85% of the 28-day cube strength after one year.

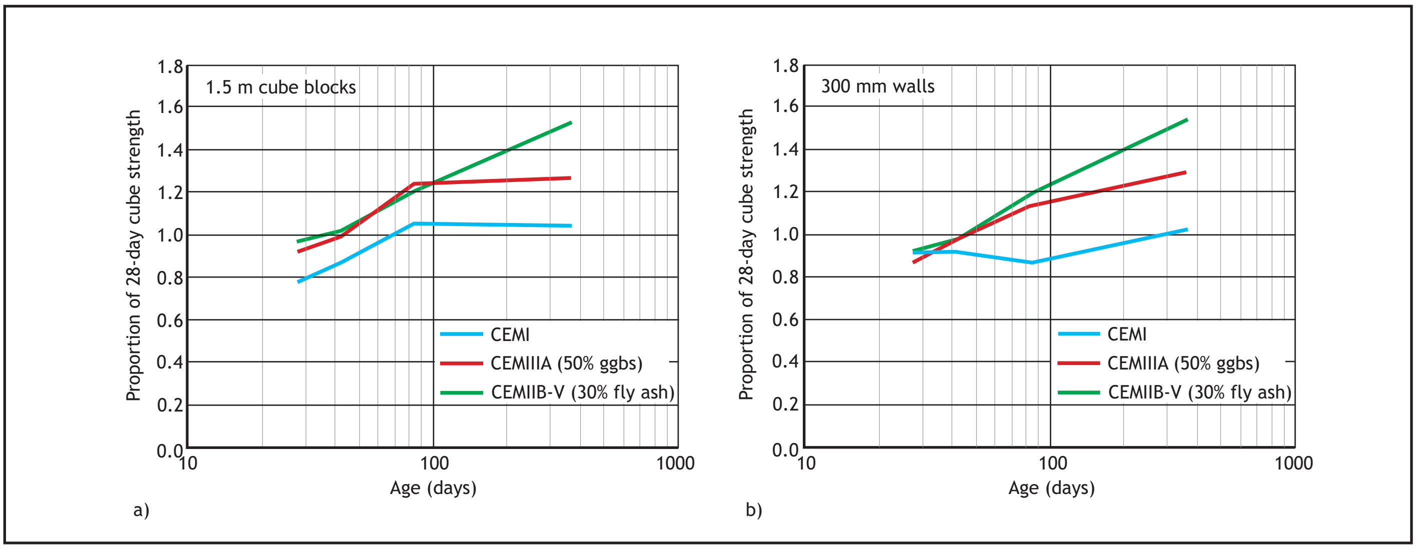


Figure 3
Strength development measured from cores.

Examples of long-term strength development are shown in Figure 3. These were obtained by testing 1:1 cores extracted from both 1.5m cubes and 300mm walls stored externally[4] and values are expressed as a proportion of the 28-day cube strength. While the long-term strength of the CEM I concrete only marginally exceeded the 28-day cube strength after one year, concrete using CEM IIIA was more than 20% higher and concrete using CEM IIB-V was more than 40% higher, indicating the long-term benefits afforded by the use of such cement types provided conditions are sufficiently moist for the hydration process to continue.

Computer models based on maturity calculations are available to predict the rate of strength development if necessary. The producer can provide basic information, for example, cement type, class and content, and the adiabatic temperature rise curve, depending upon which model is being used. The models assume that there is sufficient water for hydration to continue without interruption and this is a reasonable assumption for the first few days after casting. The validity of this assumption for longer-term predictions needs to be assessed on a case-by-case basis.

3.8 Fatigue strength

For the verification of concrete in compression or shear under cyclic loading, the design fatigue strength, $f_{cd,fat}$, is calculated using the expression:

EC2 6.76

$$f_{cd,\,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left[1 - \frac{f_{ck}}{250} \right] \quad (6)$$

where

$\beta_{cc}(t_0)$ is a coefficient for concrete strength at first load application

t_0 is the time of the start of cyclic loading

k_1 is a coefficient defined in the UK National Annex = 0.85.

The method of verification is described in BS EN 1992-1-1, Clause 6.8.7.

4. Tensile strength

In design, tensile strength is used in both serviceability and ultimate limit state calculations, for example:

- In general, considerations of cracking, shear, punching shear, bond and anchorage.
- The evaluation of the cracking moment for prestressed elements.
- The design of reinforcement to control crack width and spacing resulting from restrained early-age thermal contraction.
- Developing moment-curvature diagrams and in the calculation of deflection. In the calculation of deflection, higher tensile strengths lead to lower levels of cracking and lower deflection.
- The design of fibre-reinforced concrete.
- It is also used in the design of unreinforced concrete sections, for example, concrete pavements.

It should be noted that increasing the tensile strength may not necessarily be advantageous. For example, in the case of early thermal cracking, higher tensile strength requires an increased minimum steel ratio to accommodate the higher stress transferred to the steel when a crack occurs. In addition higher strength normally requires concrete with a higher binder content and hence higher temperature rise and thermal strain.

4.1 How tensile strength is dealt with in BS EN 1992-1-1

Tensile strength is commonly defined in one of three ways: direct tensile strength, tensile splitting strength or flexural strength. Values derived from BS EN 1992-1-1 are shown in Table 2.

Table 2
Values of tensile strength in relation to strength class.

Mix designation	C12/16	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75	C70/85	C80/95	C90/105
Mean axial tensile strength f_{ctm}	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.8	5.0
Mean splitting tensile strength $f_{ctm, sp}$	1.7	2.1	2.5	2.8	3.2	3.6	3.9	4.2	4.5	4.7	4.8	5.1	5.4	5.6
Mean flexural tensile strength $f_{ctm, fl}$	2.4	2.9	3.3	3.8	4.3	4.8	5.3	5.7	6.1	6.3	6.5	6.9	7.3	7.6

4.1.1 Tensile strength used in design

EC2 3.16

The design tensile strength of concrete, f_{ctd}, according to BS EN 1992-1-1 is taken as:

$$f_{ctd} = \alpha_{ct} f_{ctk\ 0.05}/\gamma_c \tag{7}$$

where

$f_{ctk\ 0.05}$ = characteristic tensile strength of concrete at 28 days

γ_c = partial (safety) factor for concrete = 1.5

α_{ct} = coefficient taking account of long-term effects on the tensile strength, this is an NDP with a recommended value of 1.

In BS EN 1992-1-1, the term 'tensile strength' refers to the highest average stress reached under concentric tensile loading.

For normal structural uses, the mean tensile strength, f_{ctm}, is related to the cylinder strength by the expressions:

EC2 Table 3.1

Strength classes ≤ C50/60 $\quad f_{ctm} = 0.30\ f_{ck}^{(2/3)}$ MPa (8)

Strength classes > C50/60 $\quad f_{ctm} = 2.12\ \log_e [1 + (f_{cm})/10]$ MPa (9)

Note that for strength classes ≤ C50/60 f_{ctm} is derived from f_{ck} while for the higher-strength classes > C50/60 the tensile strength is derived from f_{cm}.

4.1.2 Tensile splitting strength

The direct tensile strength is a value that is rarely determined by testing and there is no European or International Standard. However, where the tensile strength is determined by the tensile splitting test in accordance with BS EN 12390-6, BS EN 1992-1-1 permits the tensile strength to be calculated from the tensile splitting strength, $f_{ct,sp}$ as follows:

EC2 3.3

$$f_{ct} = 0.90\ f_{ct,sp} \tag{10}$$

When using this approach, tests should be on concrete achieving the target mean compressive strength, as this will result in the best estimate of the mean tensile strength.

4.1.3 Flexural tensile strength

The flexural tensile strength can be measured using the BS EN 12390-5 four-point method test procedure. It can also be calculated from the mean tensile strength by the following expressions.

The flexural strength is the higher of:

EC2 3.23

$$f_{ctm,fl} = (1.6 - h/1000)\, f_{ctm} \quad (11)$$

where
h is the total member depth in mm

or $$f_{ctm,fl} = f_{ctm} \quad (12)$$

Rearranging Expression (11), the f_{ctm} may be estimated from the flexural strength measured on a 100 × 100mm prism in accordance with BS EN 12390-5 and

$$f_{ctm} = f_{ctm,fl}/1.5$$

4.1.4 Effect of age

BS EN 1992-1-1 provides expressions for calculating tensile strength at different maturities:

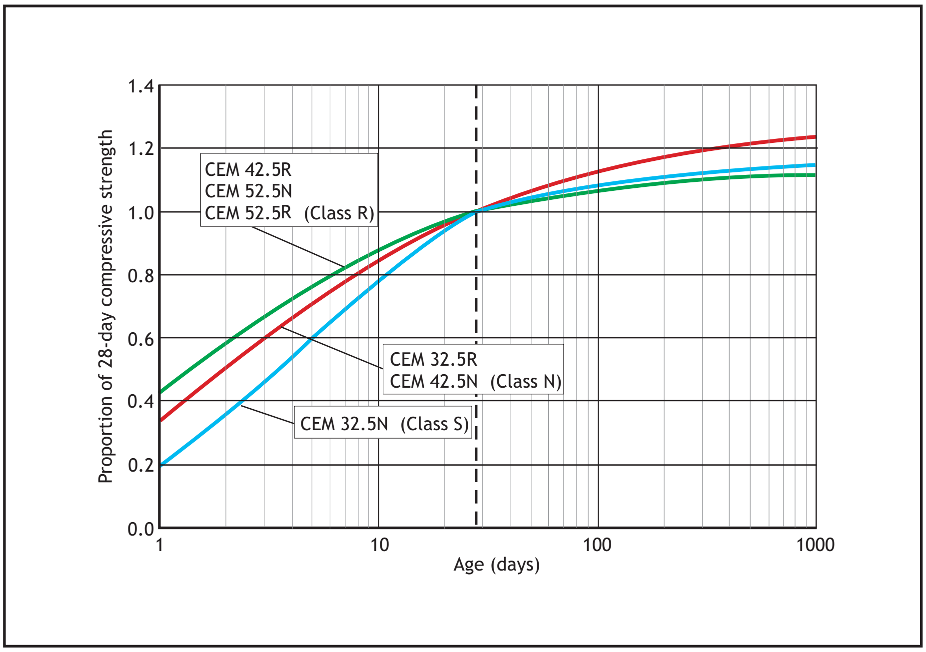
EC2 3.4

$$f_{ctm}(t) = [\beta_{cc}(t)]^{\alpha} f_{ctm} \quad (13)$$

where
$\beta_{cc}(t)$ is defined in Expression (5)
$\alpha = 1$ for $t < 28$ days
$\alpha = 2/3$ for $t \geq 28$ days.

Hence up to 28 days the development of tensile strength is the same as that of compressive strength. However, beyond 28 days the tensile strength is assumed to develop to a lesser extent as shown in Figure 4.

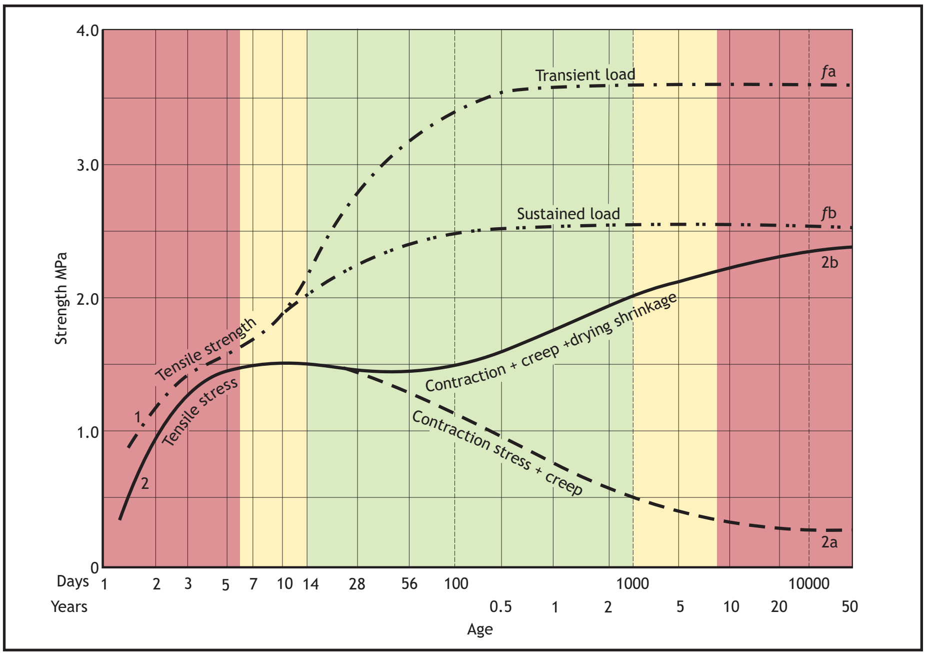
Figure 4
Rate of tensile strength development at 20°C for different cement strength classes.



When estimating development of tensile strength, for example, for the assessment of the risk of early-age cracking and the requirement for crack control steel[6], BS EN 1992-1-1 recommends that tests should be carried out, taking into account the exposure conditions and the dimensions of the structural member. For practical reasons, the test itself may not reflect directly the exposure conditions and dimensions of a structural member, but it may be possible to test specimens with compatible maturity.

In Figure 5, which shows the development of tensile strength and of tensile stress from restrained movement, the upper two lines show the tensile strength of the concrete, with the lower of these lines (f_b) reflecting the 0.7 reduction factor for a sustained load. The lower two lines show the induced early-age contraction stresses with relief from creep. The upper of the two lines (2b) shows the additional effect of long-term drying shrinkage. It can be seen that in addition to the risk of restrained early-age cracking, there is a risk of cracking from long-term drying shrinkage after ten years. This concept is simplistic as the effect of temperature which can be significant is not shown here.

Figure 5
Development of tensile strength and tensile stress from restrained movement.

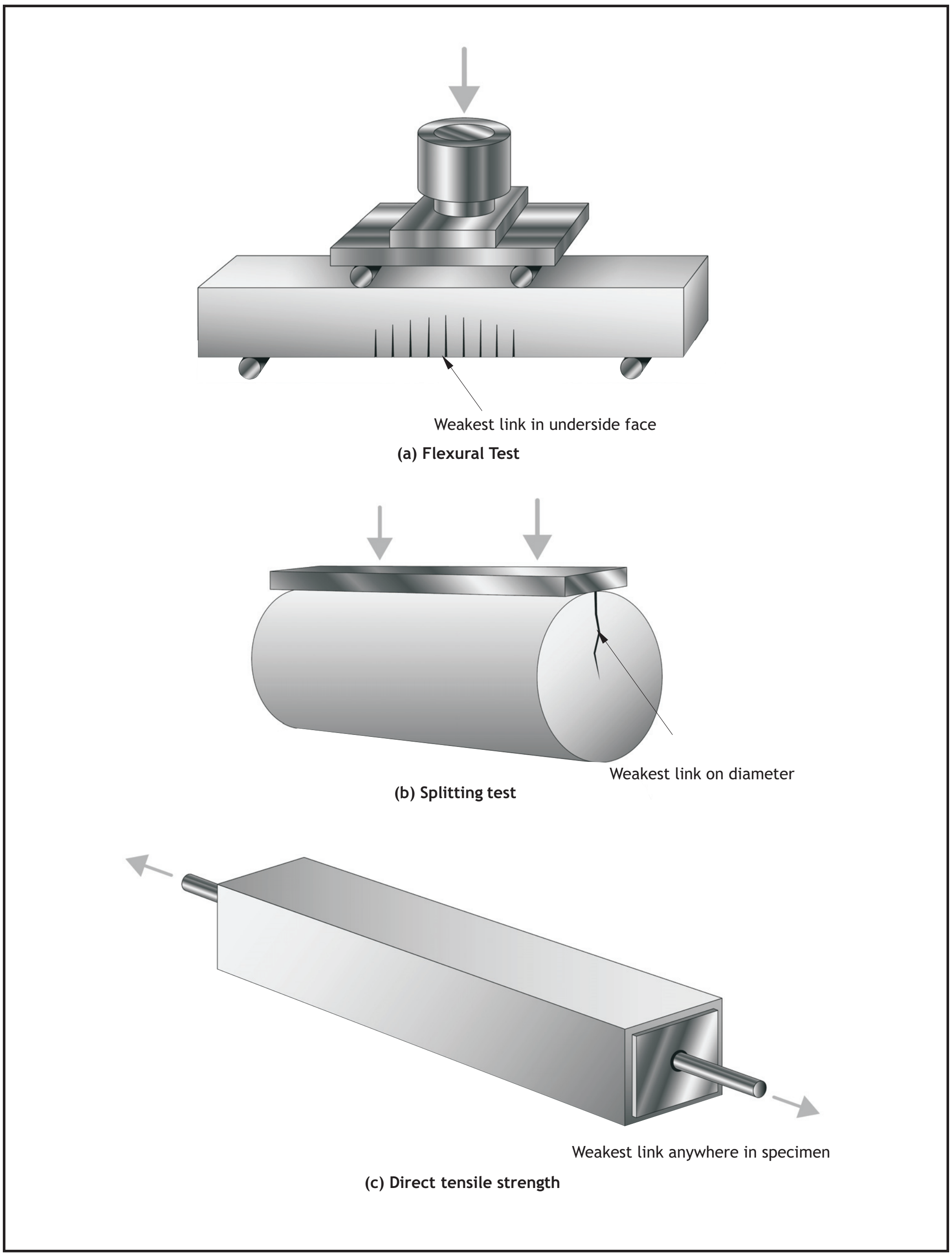


4.2 Comparison of the test methods

The tensile splitting strength should be determined from BS EN 12390-6, and the flexural tensile strength from BS EN 12390-5 using the four-point method. The alternative method of loading (centre-point loading) has been found to give results 13% higher than the reference method. Neither BS EN 12390-5 nor BS EN 12390-6 includes information on the precision of the tests.

It has been seen that different values are obtained from the different test methods, see Section 4.1. This is partly explained by the 'weakest link' concept which supposes that a tensile failure will start at the weakest point, and then propagate rapidly through the cross-section. With a larger area in tension, there is a greater probability of there being a 'weaker link' than with a smaller area, and consequently the measured tensile strength will also be lower, see Figure 6.

Figure 6
Location of the weakest link in (a) the flexural test, (b) the tensile splitting test, and (c) the direct tensile test.



While it is possible to get relatively low testing errors under laboratory conditions,[7] the use of normal compressive testing machines calibrated for cube testing may nevertheless give unreliable results. It is has been reported[5] that the coefficient of variation for tensile testing may be twice that for cube testing, for example, 6.5% compared with 3.2%[8] and that to achieve a reasonable chance of conformity, the concrete producer's design margin should be sufficient to give a failure rate of appreciably less than 1%. It has also been suggested[7] that the tensile splitting test is unsuitable as a conformity test for concrete.

The tensile values given in table 3.1 of BS EN 1992-1-1 reflect this high coefficient of variation (approximately 18%).

4.3 Some testing practical advice

Because of high test variability of tensile testing, it is recognised that compliance should be based on the measurement of compressive strength. However, specifiers may request information on the relationship between tensile and compressive strength for a particular concrete for comparison with that given in BS EN 1992-1-1.

Where information on the development of tensile strength with time for a specific concrete is sought, the test method needs to be agreed and then specified. It is recommended that either the tensile splitting test BS EN 12390-6, or the flexural test using the BS EN 12390-5 reference method is used. Due to testing variability, at least three and ideally six specimens should be tested at each age. To compensate for the lack of precision data, it is recommended that the result is presented as a mean value, rather than as individual results.

Depending on what the data are required for, the concrete mix proportions for the tests should be either:

- those that are expected to give the target mean compressive strength; the average test value is then taken as the corresponding mean tensile strength, or
- those that are expected to give the characteristic compressive strength; the average test value is then taken as the corresponding characteristic tensile strength.

4.4 Factors influencing tensile strength

Depending upon the specific requirements it may be desirable to either increase or to decrease the tensile strength. For example, to resist cracking a high tensile strength is desirable, but if cracking is likely to occur then the minimum reinforcing steel ratio may be reduced for a lower tensile strength. Factors which have an effect on the tensile strength are as follows:

- Compressive strength: in general the tensile strength varies in proportion to the compressive strength.
- The relative volumes of cement paste and aggregate have little effect on tensile strength.[9]
- Coarse aggregate type: concrete containing high-quality crushed rock coarse aggregate tends to have higher tensile strength than concrete made with gravels. However, crushed flint gravels in particular may result in a low tensile strength due to poor bond with the glassy flint surfaces.
- Aggregate size: the tensile strength tends to be higher when using smaller aggregate due to the increase in aggregate surface area and hence reduction in aggregate–cement paste bond stress.

Steel fibres do not change the tensile strength of concrete itself, but in concrete elements they control cracking and can contribute to ductile behaviour. Polymer fibres help to control cracking of concrete in the plastic state only.

5. Bond strength

In reinforcement design, BS EN 1992-1-1 covers only the use of ribbed, high-yield bars. Knowledge of the bond strength of reinforcement is required for two principal reasons:

- To establish anchor and lap lengths.
- To enable crack spacing and crack width to be calculated.

5.1 How bond strength is dealt with in BS EN 1992-1-1

EC2 8.2

BS EN 1992-1-1 provides information on bond in relation to anchor lengths. The ultimate bond stress is given by the expression:

$$f_{bd} = 2.25\,\eta_1\,\eta_2\,f_{ctd} \tag{14}$$

where

f_{bd} is the ultimate (design) bond stress

η_1 is a coefficient related to the quality of the bond condition and the position of the bar during concreting
= 1.0 for condition of good bond
= 0.7 for all other cases and for bars in structural elements built with slipforms

η_2 is related to bar diameter
= 1.0 for $\phi \leq$ 40mm (NDP)
= $(140 - \phi)/100$ for $\phi >$ 40mm

f_{ctd} is the design tensile strength defined as:

$$f_{ctd} = \alpha_{ct\ fctk,0.05}/\gamma_c \tag{15}$$

where

γ_c is the partial safety factor for concrete = 1.5

α_{ct} is a coefficient taking account of long-term effects on the tensile strength and unfavourable effects resulting from the way the load is applied = 1 (NDP).

5.2 How to control crack widths using BS EN 1992-3 and BS EN 1992-1-1

BS EN 1992-3 deals with the design of liquid-retaining and containment structures. A specific requirement of such structures is the control of crack widths to minimise or prevent leakage. The crack width is estimated from the product of the magnitude of the restrained component of contraction (early-age thermal plus shrinkage) and the crack spacing. Flexural crack spacing is determined using the expression:

EC2 7.11

$$s_{r,\,max} = 3.4c + 0.425\left(\frac{k_1\phi}{\rho_{p,\,eff}}\right) \tag{16}$$

where

c is the cover to reinforcement

ϕ is the bar diameter

$\rho_{p,eff}$ is the ratio of the area of reinforcement to the effective area of concrete

The coefficients 3.4 and 0.425 are the UK's NDPs

k_1 is a coefficient which takes account of the bond properties of the reinforcement = 0.8 for high bond bars.

The coefficient k_1 has replaced the ratio f_{ct}/f_b (= 0.67) used previously in the estimation of crack spacing in BS 8007. Other more significant changes in BS EN 1992-1-1 compared with BS 8007, most notably a reduction in the effective area of concrete in tension surrounding the steel, have led to the required area of reinforcement for crack control being significantly reduced.

Observations of early-age cracking suggest that the requirements of BS 8007 were generally applicable, with occasional crack widths in excess of those predicted.[5] On this basis it would be unacceptable to adopt a significantly less robust design. It is therefore recommended in CIRIA C660[6] that the factor of 0.7 (BS EN 1992-1-1, for use in conditions where it cannot be shown that good bond exists) should be applied to k_1, increasing the value to 0.8/0.7 = 1.14 until experience with application to early-age thermal cracking indicates that a value of 0.8 is acceptable.

5.3 Measuring bond strength

Bond testing is covered by BS EN 10080. The test required by the UK National Annex involves four-point bending of a test beam which consists of two half beams with the test bar in the tensile zone. This has replaced the previous pull-out test. The relationship between force and slip is measured and the bond strength is commonly defined as the calculated stress at which a particular magnitude of slip occurs.

5.4 Factors influencing bond strength

The bond strength is determined by the characteristics of both the reinforcement and the concrete as follows:

- For deformed bars the projected rib area has a dominant effect and BS 4449 gives minimum requirements.
- With regard to the concrete, as shown in Expression (14), the bond is related to the tensile strength and will therefore be influenced by the same factors (Section 4.4).

6. Modulus of elasticity

The value of the modulus of elasticity, *E*-value, chosen in design is fundamental to all analysis with regard to stiffness of members. For example, it is used in the calculation of:

- deflection – often the controlling factor in slab design
- moment analysis
- requirements for prestressed elements
- column shortening under load
- stresses due to restrained movements.

Such movements are also influenced by creep which is dealt with in Section 8.

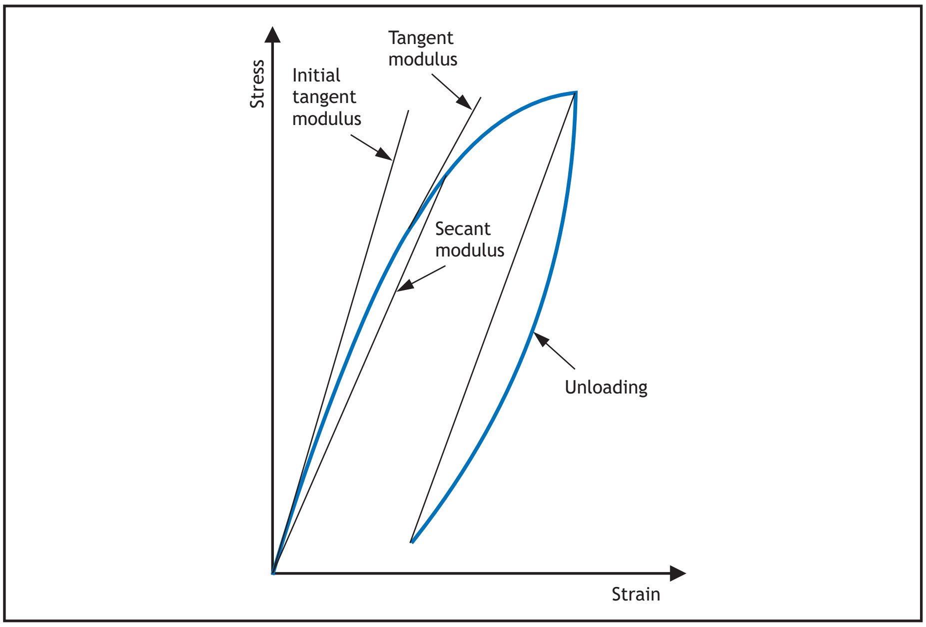
6.1 Definitions

There are two types of elastic modulus. The **static** modulus is measured by plotting the deformation of a cylinder under an applied load, usually 30–40% of the ultimate load. The **dynamic** modulus is determined by resonance methods or by the measurement of ultrasonic pulse velocity (UPV). The two test procedures do not give the same measured value of the modulus. Static modulus is the value usually quoted by concrete producers.

The *E*-value is the ratio between stress (load/area) and strain (deformation, or change of length/length). As concrete is not a truly elastic material, the relationship between stress and strain is not constant. Three *E*-value conventions are used:

- the secant modulus
- the tangent modulus
- the initial tangent modulus (see Figure 7).

Figure 7
Diagrammatic stress–strain relationships for concrete.[11]



These are all measurements of the static modulus. The initial tangent modulus is also approximately equal to the dynamic modulus and, by definition, is only applicable at very low stress levels. The most generally useful measure is the secant modulus, and in BS EN 1992-1-1 it is the secant modulus, Ecm, that is used in design.

6.2 How *E*-value is dealt with in BS EN 1992-1-1

6.2.1 Use of *E*-value in design

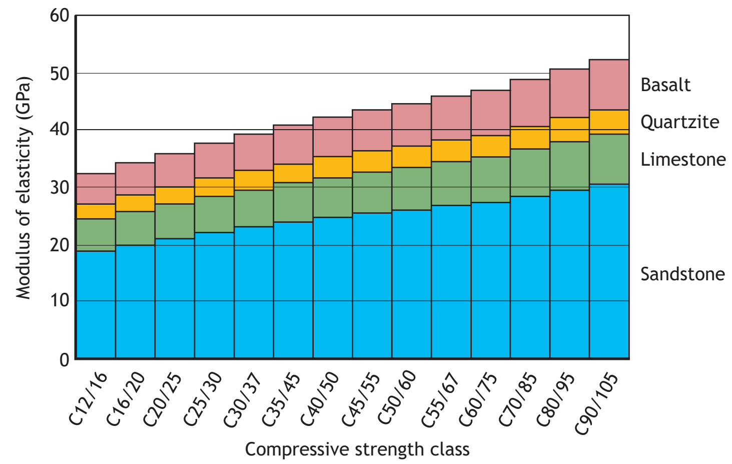
In design, the secant modulus, E_{cm} (in GPa), is derived from the mean compressive strength, f_{cm} (in MPa), from the expression:

EC2 Table 3.1

$$E_{cm} = 22\,[f_{cm}/10]^{0.3}\ \text{GPa} \qquad (17)$$

In Figure 8 moduli derived from Expression (17) are secant values for concrete loaded from $\sigma_c = 0$ to $0.4f_{cm}$ with quartzite aggregates. For limestone and sandstone aggregates, the value is reduced by 10% and 30% respectively and for basalt aggregates it is increased by 20%.

Figure 8
Modulus of elasticity in relation to compressive strength class and aggregate type.



Although not explicitly stated in BS EN 1992-1-1, Clause 3.1.3(2), the expression for quartzite aggregates may also be applied to concretes with siliceous aggregates. This approach assumes that the designer knows the aggregate to be used, however this may not be the case until the concrete supplier is selected. In contrast, in the case of very high strength concrete the type of course aggregate is usually known and often specified.

When the elastic modulus is critical to the performance of a structure then testing is recommended.

In the design process E is applied as follows:

- For serviceability calculations the mean value E_{cm} is used.
- For ultimate limit state calculations a partial safety factor, γ_{cE}, is used to give a design value for the modulus, $E_{cd} = E_{cm}/\gamma_{cE}$ (where γ_{cE} is 1.2).
- For long-term deflection calculations E_{cm} is modified by creep to give an effective modulus, $E_{c,eff}$. This is calculated using the expression $E_{c,eff} = E_{cm}/(1 + \varphi)$ where φ is the creep coefficient with a value typically between 1 and 3 (Section 8.1).

Poisson's ratio is also used in elastic analysis and in accordance with BS EN 1992-1-1 is taken as 0.2 for uncracked concrete and 0 for cracked concrete.

6.2.2 Variation with age

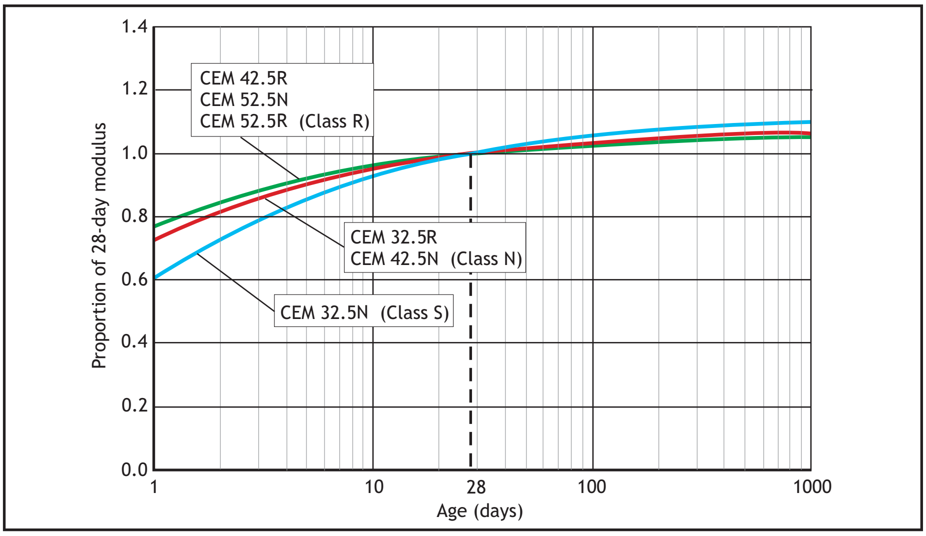
EC2 3.5

The variation of modulus of elasticity with time is estimated using the expression:

$$E_{cm}(t) = [f_{cm}(t)/f_{cm}]^{0.3}\,E_{cm} \qquad (18)$$

where $E_{cm}(t)$ and $f_{cm}(t)$ are the values at an age of t days and E_{cm} and f_{cm} are the values at 28 days. The rate of development of modulus of elasticity is shown in Figure 9. It is apparent that modulus develops more rapidly than strength in the very short term, with less significant growth beyond 28 days. In addition the cement type has much less of an effect. This is not surprising as the usually stiffer aggregate comprises about 70% of the volume of the concrete and is therefore the dominant factor.

Figure 9
Rate of development of modulus of elasticity at 20°C for different cement strength classes.



6.3 Measuring the *E*-value

6.3.1 Test methods

Work is in progress within the European Committee for Standardisation (CEN) to develop a test procedure to measure the static modulus of elasticity. While there may be some preliminary loading cycles to remove the effect of creep, the value from this test is usually taken as being the approximate secant modulus. This static modulus test will be published in the BS EN 12390 series.

In the ASTM C 512-02 creep test, the *E*-value is determined from the strain at first loading. As it is based on the difference between only two measurements, it may be less reliable than that obtained using the BS EN 12390 test.

The **initial tangent modulus** may be determined in two ways:

1. By undertaking ultrasonic pulse velocity (UPV) measurements in accordance with BS EN 12504-4. However, there is no procedure for converting the UPV readings into an initial tangent modulus. The procedure is covered in BS 1881-209 and it is expected that this procedure will be included in the UK National Annex to BS EN 12504-4.
2. Measuring the dynamic modulus by means of a variable frequency oscillator. The procedure for measuring the dynamic modulus (≈ initial tangent modulus) is given in BS 1881-209.

6.3.2 Guidance on *E*-value testing

As deflection forms part of the serviceability limit state, mean *E*-value is appropriate and so the concrete mix proportions used for testing should be those that target a mean compressive strength.

Care is needed when selecting a test machine to use for *E*-value tests. Machines that are in calibration for cube testing may not be suitable for modulus testing. The problems tend to be with high-capacity machines (heavy platens) and machines where the ball seating is not free to rotate. The indication of a problem may be identified if there are large differences between the three strain readings.

When a measured *E*-value is being used, the designer could consider using a reduced partial safety factor of γ_{cE}, say 1.1 in place of the normal 1.2, giving a higher design value. A safety factor γ_{cE} less than 1.1 is not recommended due to the uncertainty associated with the measured value and variability of production.

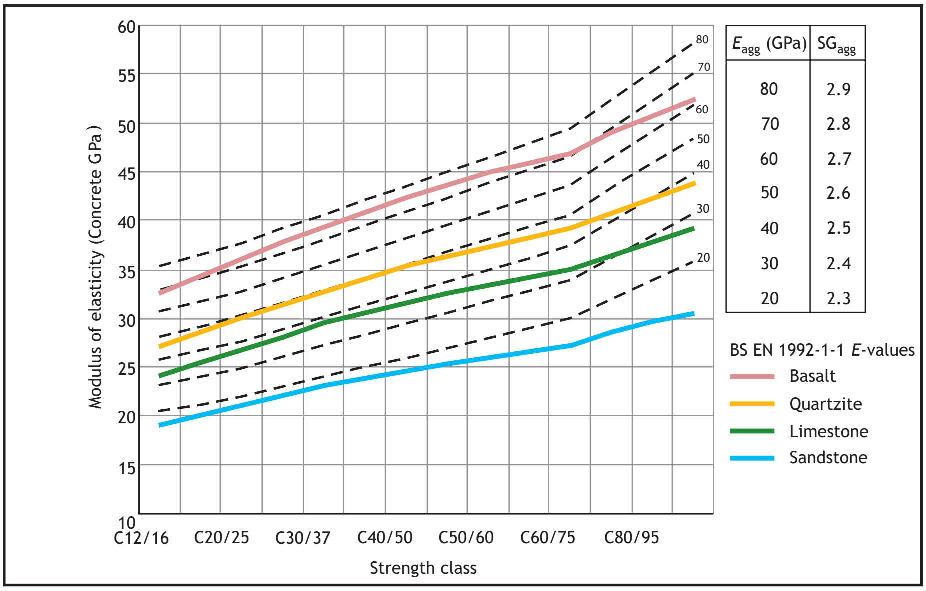
6.4 Factors influencing modulus of elasticity

There are a number of factors to be considered:

- **Compressive strength.** While a higher strength leads to a higher modulus of elasticity, there is no direct proportionality. For example, to increase the modulus by 20% it is necessary to increase the strength by at least three strength classes which may not be a cost-effective solution.
- **Aggregate E-value.** The aggregate comprises about 70% of the volume and is usually stiffer than the cement paste. Hence the *E*-value of the aggregate has a significant effect on the *E*-value of the concrete. Figure 10 shows the *E*-values for concrete estimated according to BS EN 1992-1-1. Values are compared with predictions based on strength class and aggregate *E*-value (indicated by the dashed lines) using a model developed for concrete in nuclear applications.[12] It is unlikely that a concrete producer will have information on the aggregate modulus but it is usually accepted that it is proportional to aggregate density. An indication is also given in Figure 10 of the specific gravity (SG) of the aggregate associated with different *E*-values.[13]
- **Aggregate volume.** As the aggregate is stiffer than the cement paste, the *E*-value of the concrete may be increased by around 5%[9] by increasing the volume of aggregate. This is a small increase when compared with the effect of aggregate type, but mix design (relative volumes of aggregate and paste) is something over which the producer has control whereas aggregate type often cannot be changed easily.

- **Mineral additions.** The presence of either fly ash or slag in a concrete will result in reduced elastic deformations provided the design load is not applied at a maturity less than 28 days at 20°C and conditions are such that long-term strength gain can occur.

Figure 10
The relationship between strength class, aggregate *E*-value (and specific gravity) and concrete *E*-value.



7. Tensile strain capacity

The tensile strain capacity, ε_{ctu}, is the maximum strain that the concrete can withstand without a continuous crack forming. It is used in the strain-based approach described in CIRIA C660[6] to assess the risk of early-age thermal cracking and in the estimation of crack width.

7.1 How tensile strain capacity is dealt with in BS EN 1992-1-1

Tensile strain capacity of concrete ε_{ctu} is not dealt with in BS EN 1992-1-1. However, in a comprehensive review of published data[14] a simple linear relationship was identified between ε_{ctu} and the ratio of the tensile strength f_{ctm} to the elastic modulus E_{cm} (measured in compression) as follows:

$$\varepsilon_{ctu} = [1.01(f_{ctm}/E_{cm}) \times 10^6] + 8.4 \text{ microstrain} \qquad (19)$$

Simplifying this expression to:

$$\varepsilon_{ctu} = f_{ctm}/E_{cm} \qquad (20)$$

was found to provide a lower bound value for use in design. Using this relationship, values of ε_{ctu} have been derived from estimates given in BS EN 1992-1-1 for tensile strength (Section 4) and elastic modulus (Section 6) for each strength class and for different aggregate types.

Values estimated from BS EN 1992-1-1 apply under conditions of short-term loading. To take account of sustained loading during an early thermal cycle, two factors are applied:

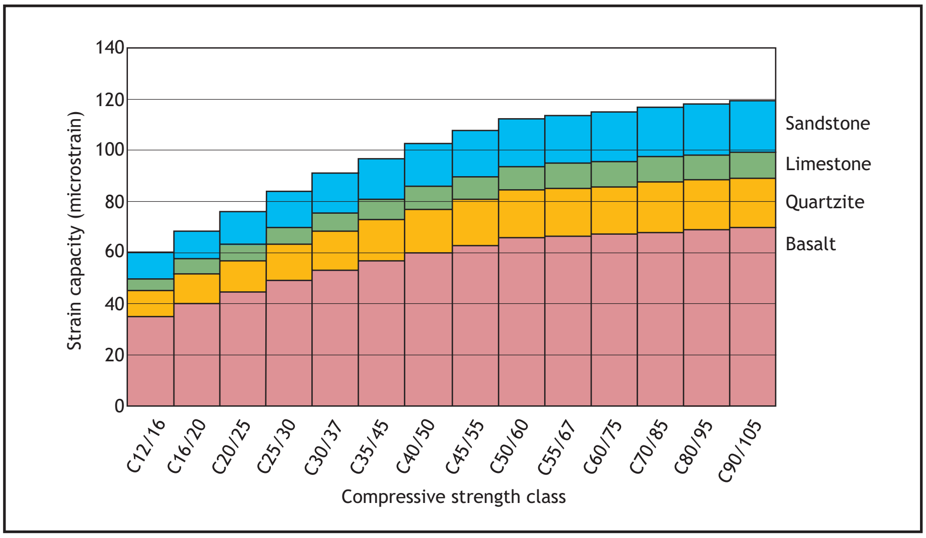
1. a creep coefficient, which increases the tensile strain with time, and
2. a coefficient to take account of reduced capacity under sustained loading.

The net effect on ε_{ctu} is an increase of 23%[6]. Results obtained on this basis are shown in Figure 11. To assess cracking at later life, ε_{ctu} may be derived using Expression (19) by applying age factors to f_{ctm} and E_{cm}, that is:

$$\varepsilon_{ctu}(t) = f_{ctm}(t)/E_{cm}(t)$$

At 28 days this gives a value 43% higher than the three-day value. The effect of the aggregate is shown in Figure 11 through the change in elastic modulus.

Figure 11
Early-age (three-day) tensile strain capacity under sustained loading.



7.2 Measuring tensile strain capacity

There is no standard test for measuring tensile strain capacity. The most direct way of measuring tensile strain capacity ε_{ctu} is to subject prisms to direct tensile loading and to measure the strain up to failure.[15] Direct measurement may also be achieved by creating conditions within a test specimen that are similar to those which lead to early thermal cracking - for example, stress rig tests subject a dog-bone-shaped specimen to a thermal cycle. During heating the concrete is allowed to expand freely but it is restrained during contraction. When the concrete cracks, the release of strain defined by the crack width is used to derive the strain at failure. This may be compared with the measurement of the temperature change and hence, with a knowledge of α_c (the coefficient of thermal expansion), the restrained thermal contraction required to cause failure may be calculated.

Direct measurement of ε_{ctu} generally requires a large test rig and relatively sophisticated monitoring equipment. An alternative approach is to derive ε_{ctu} from measurements of tensile strength f_{ctm} and elastic modulus E_{cm}.[14]

7.3 Factors influencing tensile strain capacity

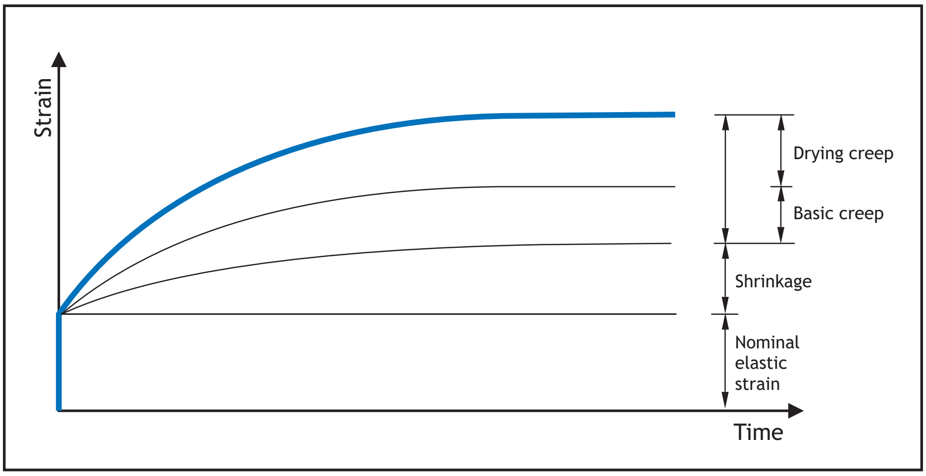
The tensile strain capacity can be regarded as the ratio of the tensile strength to the modulus of elasticity and the factors influencing these individual properties will also influence its value. The aggregate type is of particular significance as determined by its modulus of elasticity. Less stiff aggregates lead to higher tensile strain capacity.

8. Creep

Creep is time-dependent deformation (strain) under sustained loading, excluding non-load-induced deformations such as shrinkage, swelling, thermal strain, see Figure 12. Creep strain is typically two to four times the elastic strain[16] and knowledge of creep is needed for several reasons:

- To estimate long-term deflections in beams and long-term shortening in columns and walls. This may be important, for example, in establishing tolerances for movement when fixing rigid, brittle partitions to a concrete frame.
- To estimate prestress losses.
- To estimate stress relaxation and redistribution over time. This may be beneficial in reducing the risk and/or extent of cracking. Creep in tension may also partly relieve the stresses induced by other restrained movements, for example, drying shrinkage, thermal contraction; or by loading.

Figure 12
Time-dependent deformations in concrete subjected to a sustained load - change in strain of a loaded and drying specimen.[11]



8.1 How creep is dealt with in BS EN 1992-1-1

Generally, creep depends on ambient humidity, the dimensions of the element, and the composition of the concrete. It is also influenced by the maturity of the concrete when first loaded and on the duration and magnitude of the loading.

The ultimate creep strain is calculated by multiplying the elastic strain by a creep coefficient using the expression:

EC2 3.6

$$\varepsilon_{cc}(\infty, t_0) = \varphi(\infty, t_0)\,(\sigma_c/E_c) \tag{21}$$

where

$\varepsilon_{cc}(\infty, t_0)$ = creep deformation at time $t = \infty$
$\varphi(\infty, t_0)$ = creep coefficient at time $t = \infty$
σ_c = constant compressive stress applied at time $t = t_0$
E_c = tangent modulus = 1.05 E_{cm}

The creep coefficient $\varphi(\infty, t_0)$ is determined by the following factors:

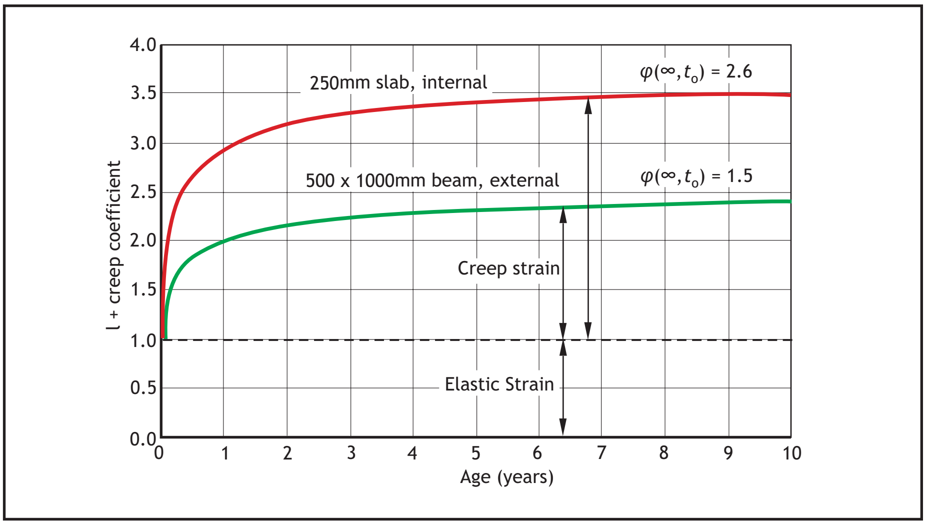
- **Relative humidity** – for indoor conditions (RH = 50%) and for outdoor conditions (RH = 80%). More creep occurs under dryer conditions.
- **Element geometry** – defined by a notional thickness which also affects the rate of drying.
- **Strength class.**
- **Age at loading** – which affects the stress/strength ratio and its change thereafter.
- **Cement class** – Slow, Normal or Rapid strength gain (Classes S, N or R), see Section 3.6.
- **Stress/strength ratio at loading** – Expression (21) only applies up to a stress/strength ratio of 0.45 based on the characteristic cylinder strength at the time of loading. Where the stress exceeds this value, micro-cracking will cause an increase in creep, and expressions are provided in BS EN 1992-1-1 for taking this creep non-linearity into account.

In order to develop creep curves showing the development with time, Appendix B (Informative) of BS EN 1992-1-1 provides an expression which takes account of relative humidity, element size, the strength class and the age at loading. Estimated creep coefficients are shown in Figure 13 for two examples:

- a 500 × 1000mm precast bridge beam using C35/45, externally exposed (80% RH) and loaded at 28 days;
- an internal floor slab (50% RH) using C30/37 loaded at 14 days.

About 50% of the ultimate creep occurs during the first few months and 90% within the first few years. The coefficient of variation using the approach of BS 1992-1-1 is declared to be 20%.

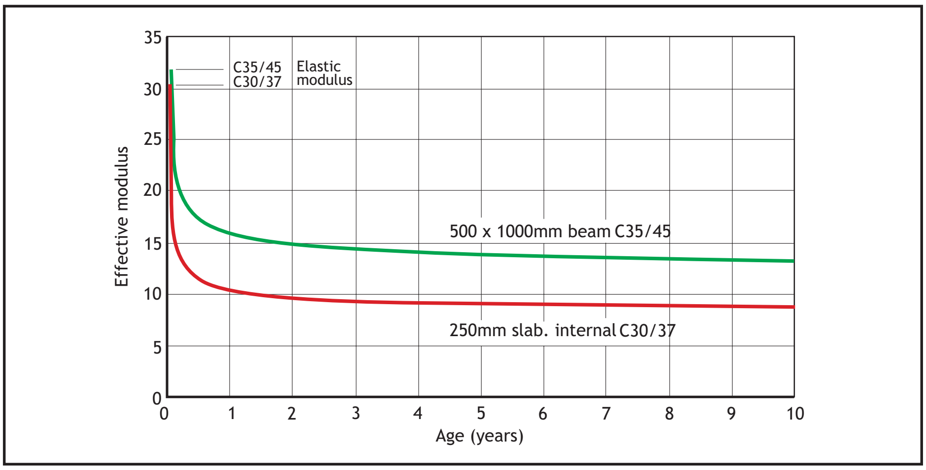
Figure 13
Estimates of creep coefficients, φ.



Values of effective modulus, $E_{c,eff}$, calculated using the creep coefficients shown in Figure 13 are shown in Figure 14.

While the mechanisms of tensile creep and compressive creep may be different, it is normal in design to assume that the creep coefficients in tension and compression are the same. [1]

Figure 14
Estimates of effective modulus, $E_{c,eff}$.



8.2 Measurement of creep

8.2.1 Test methods

There is currently no European standard test for creep of concrete in the BS EN 12390 series, but a test is being developed for repair products. This test is defined in BS EN 13584-2 and uses 40 × 40 × 160mm prisms which makes it unsuitable for most normal concretes. Work has started on an ISO test (ISO/ DIS 1920-9) and this is at the draft international standard stage.

ASTM C 512 provides a method of measuring the total creep (basic creep plus drying creep) of concrete. While some standard conditions are defined, it is recognised that these may be varied to obtain information relevant to a specific project. Six 150mm diameter cylinders are cast, two used for strength testing, two used for creep testing and two are left without loading to determine the changes of strain without load, for example, those due to drying and to autogenous shrinkage. The applied load shall not be more than 40% of the compressive strength at the time of loading. Readings are taken immediately the load is applied, then again 2–6 hours later, and then at defined intervals until they have been loaded for one year. A procedure is given for calculating the creep rate.

According to Brooks,[17] the equipment for the ASTM C 512 test is large and expensive and researchers tend to use smaller, less expensive equipment.

8.2.2 Guidance on creep testing

For normal indoor conditions where project-specific data are required, the standard requirements of ASTM C 512 may apply, and a stress/strength ratio of 40% may be used. However, recognising the many variables that affect creep, it is recommended that tests be undertaken under conditions that replicate as closely as possible those likely to occur in practice. Particular consideration should be given to the following:

- Achieving a representative concrete and compressive strength.
- Loading at a representative age and at a representative stress.

- Achieving representative drying conditions. Drying is a function of surface-to-volume ratio and it is not normally practical to vary the specimen size. It may be necessary to take a worse case, or test at a different relative humidity and to interpolate for different parts of the section.
- Achieving a representative temperature. In the normal range of operating temperatures of structures, the effect of temperature is relatively small, but it may need to be considered for specific applications.
- Continuing the test for a sufficiently long period to achieve reasonably reliable extrapolation for the life of the structure. Gilbert[16] has reviewed the mathematical expressions for the shape of the creep coefficient versus time curves, and identified the more useful expressions. He also concluded that the expressions for predicting ultimate creep from 28-day creep test data were not reliable and a longer testing period is recommended. Testing of concretes for nuclear pressure vessels[18] identified that three months was acceptable, representing about half a 30-year period when expressed on a log timescale.

Expression (1) of ASTM C 512 should be used to calculate the creep rate, and the creep deformation at, say, 30 years. The expression may also be used to calculate the creep coefficient (as opposed to assuming it, as in BS EN 1992-1-1) by dividing the creep deformation by the measured elastic strain.

8.3 Factors influencing creep

Factors affecting creep, other than those already included in the model of BS EN 1992-1-1, are as follows:

- Aggregate volume. As creep takes place in the cement paste, an increase in the volume of the aggregates will reduce creep.
- The type of cement is important if the age of loading is fixed. Cements that hydrate more rapidly will have higher strength at the age of loading, a lower stress/strength ratio and a lower creep. However, where the stress/strength ratio is the same at loading and the environment is one where the strength will continue to develop, cements that develop more strength after loading will have a lower creep. This explains, at least in part, why under some circumstances concretes containing CEM II (Portland fly ash cement) or CEM III (blastfurnace cement) tend to have lower creep.[19]
- The presence of reinforcement can significantly reduce creep and this should be taken into account during the design process. This aspect of reducing creep is not under the control of the concrete producer.
- Since creep deformation is a function of the *E*-value of the concrete, the factors affecting the modulus of elasticity will also affect creep strain.

9. Shrinkage

For design purposes, shrinkage is a combination of autogenous shrinkage and drying shrinkage. While it is recognised that shrinkage may occur while concrete is in its plastic state, these deformations are not considered within the design process.

Knowledge of shrinkage is important for several reasons:

- If shrinkage is restrained, cracking may occur and the concrete will require adequate reinforcement to limit crack widths.
- In prestressed concrete, shrinkage will result in loss of prestress.
- In asymmetrically reinforced concrete, deflections will increase.
- Axially loaded columns or walls may be subject to increased shortening.
- Creep may be increased with increased shrinkage.

9.1 Types of shrinkage

Autogenous shrinkage, ε_{ca}, occurs during early hydration and is caused by the internal consumption of water during hydration, the hydration products occupying less volume than the unhydrated cement and water. Historically, autogenous shrinkage in normal structural concrete was assumed to be of low magnitude (<100 microstrain) and has been ignored in design. However the tensile strain capacity of the concrete is only of the order of 100 microstrain, see Section 7, hence, in relation to the risk of cracking of restrained concrete, even this small strain may be significant. BS EN 1992-1-1 assumes that some autogenous shrinkage occurs in all structural concretes. As it occurs largely during hardening, BS EN 1992-1-1 recommends that it should be considered specifically when new concrete is cast against hardened concrete, i.e. in relation to the risk of early-age cracking.

In high-strength concrete with a low w/c ratio, the autogenous shrinkage is significantly higher and may exceed the drying shrinkage.

Drying shrinkage, ε_{cd}, is caused by the loss of water from the concrete to the atmosphere. Generally this loss of water is from the cement paste, but with a few types of aggregate the main loss of water and contribution to the drying shrinkage of concrete is from the aggregate. Drying shrinkage is relatively slow and the stresses it induces when restrained are partially relieved by tensile creep.

The rate of drying shrinkage is dependent upon the relative humidity (RH) of the surrounding air and the element geometry.

9.2 How shrinkage is dealt with in BS EN 1992-1-1

EC2 3.8

In BS EN 1992-1-1, the total shrinkage is taken as the sum of the autogenous shrinkage, and the drying shrinkage:

$$\varepsilon_{cs} = \varepsilon_{ca} + \varepsilon_{cd} \qquad (22)$$

9.2.1 Autogenous shrinkage

The ultimate autogenous shrinkage is calculated from the specified characteristic cylinder strength and is given by the expression:

EC2 3.12

$$\varepsilon_{ca}(\infty) = 2.5(f_{ck} - 10) \times 10^{-6} \tag{23}$$

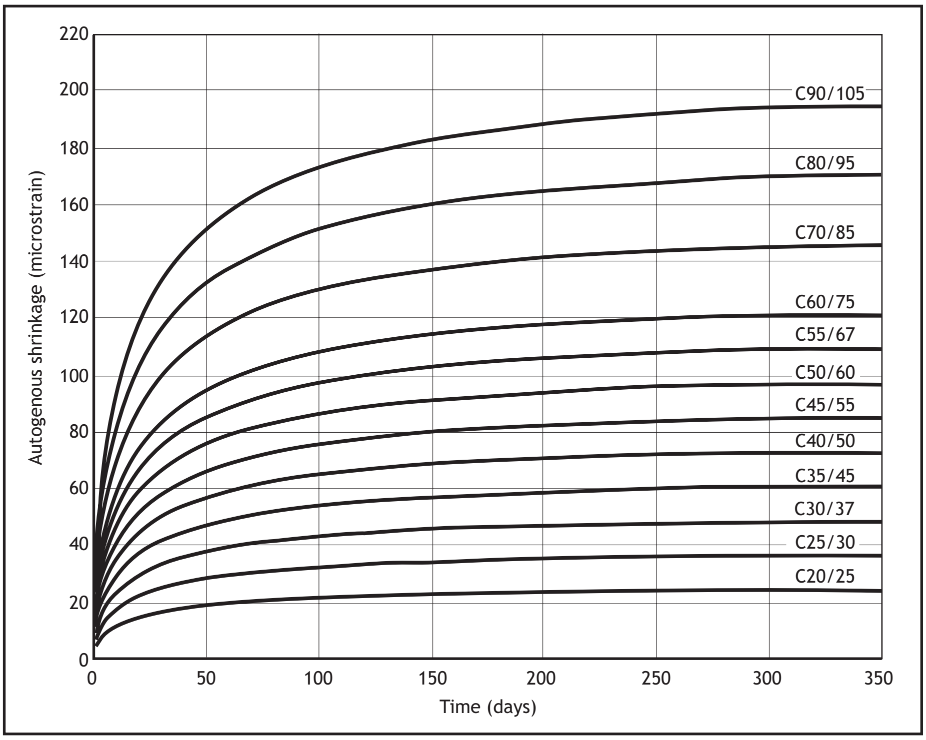
and at time, *t* days, the autogenous shrinkage is:

EC2 3.11 and 3.13

$$\varepsilon_{ca}(t) = \varepsilon_{ca}(\infty) \times [1 - \exp(-0.2\, t^{0.5})] \tag{24}$$

Design values of autogenous shrinkage estimated using Expressions (23) and (24) are shown against age in Figure 15.

Figure 15
Autogenous shrinkage in relation to strength class.



9.2.2 Drying shrinkage

The nominal unrestrained drying shrinkage is calculated by a complex expression in BS EN 1992-1-1, Annex B or by looking up values in BS EN 1992-1-1, Table 3.2. Designers should be aware that these expressions reflect old concrete technology. For example, lower w/c ratios were achieved by using more cement without the use of admixtures.

Nominal drying shrinkage for a particular element is estimated taking into account the following factors:

- strength class
- cement class
- relative humidity
- element geometry – defined by the notional thickness (2 × cross-sectional area/perimeter).

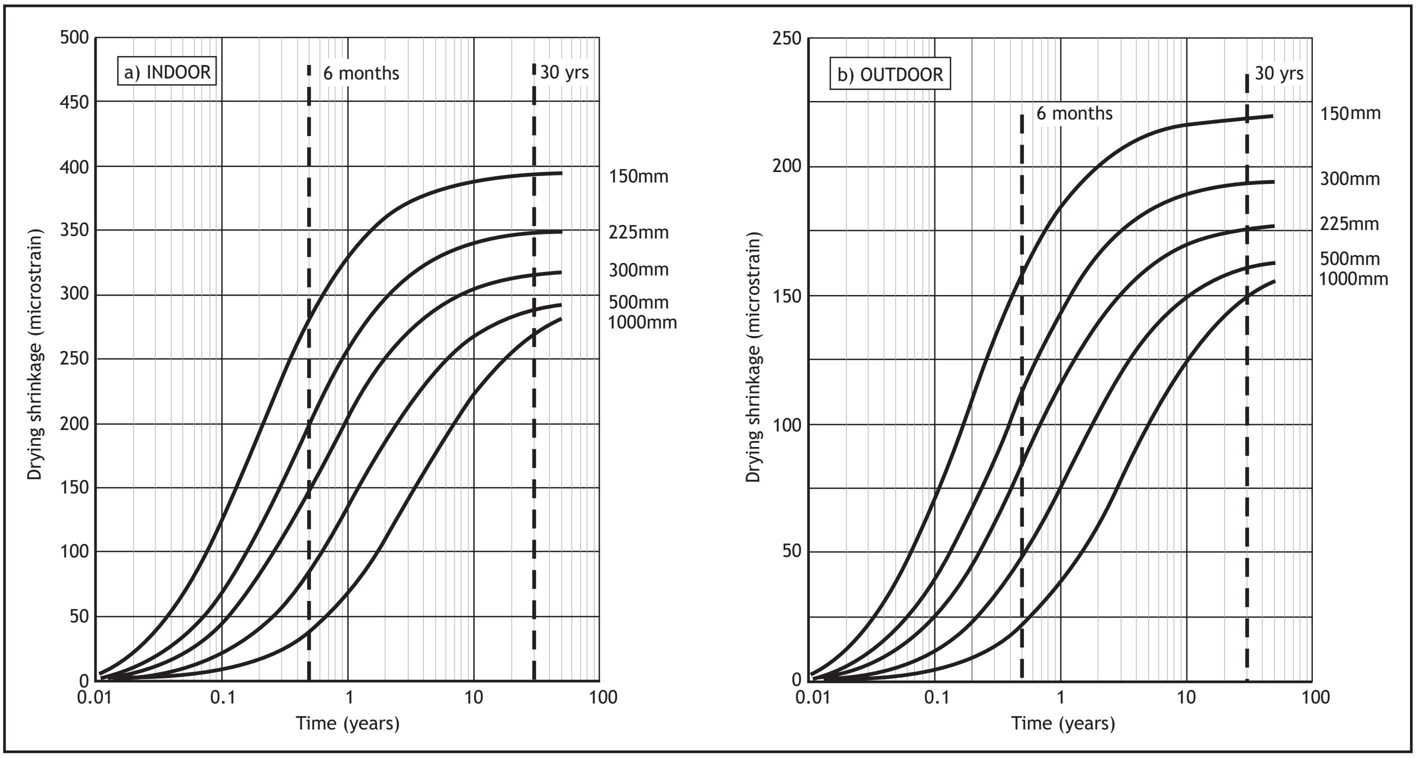


Figure 16
Drying shrinkage for (a) indoor and (b) outdoor conditions using C30/37 in sections of varying notional thickness.

Some typical values for indoor and outdoor exposure estimated using the expressions in BS EN 1992-1-1 for a range of notional thicknesses are shown in Figure 16.

While the procedures in BS EN 1992-1-1 take account of cement type, no account is taken of aggregate type. There is no recognition of the high drying shrinkage that can occur when certain aggregate types are used. However, when shrinkage is critical, it would be expected that the shrinkage of the aggregates would be assessed, see Section 9.3.2.

9.3 Measurement of shrinkage

9.3.1 Autogenous shrinkage

There is no specific European or international standard for the measurement of autogenous shrinkage. Measurement is particularly difficult as it must be undertaken immediately after casting the concrete. It must be noted however that because of the early age at which much of the autogenous shrinkage occurs, much of it is relieved by creep. A review of published data (CIRIA C660 Appendix 4[6]) has indicated that this is taken into account in the expressions in BS EN 1992-1-1 and that the values derived are those that contribute to stress development if restrained.

9.3.2 Drying shrinkage

There is no specific European test for drying shrinkage of concrete in the BS EN 12390 series. A test developed for repair products is defined in BS EN 12617-4, and uses 40 x 40 x 160 prisms, which makes it unsuitable for most normal concretes.

Any drying shrinkage test on concrete will give the total unrestrained shrinkage, i.e. the combined drying shrinkage and residual autogenous shrinkage from the start of the test. For normal-strength classes (up to C40/50), the component of residual autogenous shrinkage would be expected to be small (< 20 microstrain) but for very high-strength classes, residual autogenous shrinkage may dominate. Hence, for high-strength concrete, an allowance for the autogenous shrinkage (which takes place up to the age at which the initial datum reading is taken) should be added to the drying shrinkage test value, to give a total shrinkage value for use in design.

As drying shrinkage is related to the serviceability limit state, the concrete mix proportions used for testing this property should be those that are expected to give the target mean compressive strength. If the drying shrinkage test uses the relative humidity that is of interest, the values obtained in the short-term test can be inserted into Expression 3.9 in BS EN 1992-1-1 and the basic (unrestrained by reinforcement) drying shrinkage strain calculated. By assuming proportional changes, it is also possible to estimate the drying shrinkage at other relative humidities.

Work has started on an ISO test (ISO/DIS 1920-8). This is at the draft international standard stage and is based on an Australian test procedure (AS 1012.13-1995).

The AS 1012.13 test method for measuring drying shrinkage of concrete uses three 75mm prisms that are 285mm long. After 24 hours in the mould, prisms are conditioned in lime-saturated water for seven days at 23 ± 2°C, after which time the length of each specimen is measured to an accuracy of 0.001mm (the datum reading). They are then stored in a chamber at 23 ± 2°C and 50 ± 4% relative humidity for eight weeks, with length readings being taken at regular intervals in the first week and weekly thereafter. The rate of drying shrinkage is a function of the specimen volume-to-surface-area ratio. A typical shrinkage value after eight weeks drying is 750 microstrain (0.075%).

The drying shrinkage of aggregates is measured on concrete using the BS EN 1367-4 test. In the UK, in areas where aggregates with high drying shrinkage occur, BS 8500-2 places a drying shrinkage limit of 750 microstrain on the aggregates. The designer may relax this requirement, but they would be expected to take any resulting higher shrinkage into account. The drying shrinkage obtained by this test should not be taken as being the basic (unrestrained by reinforcement) drying shrinkage strain of the concrete itself.

9.4 Factors influencing shrinkage

9.4.1 Autogenous shrinkage

The autogenous shrinkage of normal structural concrete is low (< 100 microstrain) and there may be little benefit in trying to reduce it further. With high-strength concrete made with a low water/cement ratio (< 0.40), the autogenous shrinkage may exceed the drying shrinkage. As the w/c ratio and cement content are dictated by other requirements, there may be little scope for reducing the autogenous shrinkage by adjusting these mix parameters.

Other factors which may affect autogenous shrinkage are as follows:

- The use of a small proportion of lightweight aggregate with a high absorption (for example, replacement of 6% of sand[20] will maintain a high internal humidity and reduce auto-genous shrinkage).
- There is limited evidence (summarised in reference 6) that autogenous shrinkage may be affected by the use of mineral additions. Likely changes compared with concrete using CEM I alone expressed as weight of addition on total binder content, are as follows:
  - increased by 10% for every 1% of silica fume
  - reduced in direct proportion to the mass percentage of fly ash
  - increased by 8% for every 10% of ggbs.

  It is not recommended that these changes are used in design, but where a reduction in autogenous shrinkage is desirable it may be appropriate to undertake testing to review these options.

9.4.2 Drying shrinkage

Drying shrinkage is caused by the loss of water from the cement paste and in some cases from the aggregate. In addition to the parameters included in the model of BS EN 1992-1-1, the following factors will influence drying shrinkage:

- The relative volume of the cement paste and aggregate. Reducing the cement paste volume will reduce shrinkage. This may be achieved by increasing the maximum aggregate size. Increasing the aggregate volume from 71% to 74% may reduce drying shrinkage by about 20%.[16]
- The relative stiffness of the cement paste and aggregate. The aggregate restrains shrinkage of the cement paste, so the higher the *E*-value of the aggregate the lower the shrinkage.
- Use of aggregates with a low drying shrinkage.
- Use of plasticising admixtures to achieve the required w/c ratio and consistence without increasing the cement content will reduce drying shrinkage.[9]
- Use of special admixtures that either reduce or compensate for drying shrinkage.

10. Thermal expansion

The coefficient of thermal expansion, α_c, of concrete is a measure of the free strain produced in concrete subject to a unit change in temperature and is usually expressed in microstrain per degree centigrade (με/°C). Values are typically in the range 8–13 με/°C. The occurrence of thermal strain has a number of design implications as follows:

- The need to provide joints to accommodate the movement.
- The provision of tolerances for elements attached to the concrete, for example, cladding panels.
- Design of reinforcement to control crack widths when the thermal contraction is restrained. This may be of particular concern at early age when the heat of hydration from the cement and additions (see Section 14) may lead to temperature changes up to about 50°C, and subsequent contraction on cooling can lead to early-age thermal cracking.[6]

10.1 How the coefficient of thermal expansion is dealt with in BS EN 1992-1-1

The Eurocode states that unless more accurate information is available, the coefficient of thermal expansion may be taken as 10 microstrain/°C. As shown in Table 3, occasionally this may not be a conservative value.

Table 3
Coefficients of thermal expansion of coarse aggregate and concrete[5,20].

Coarse aggregate/rock group	Thermal expansion coefficient (microstrain/°C)		
	Rock	Saturated concrete	Design value
Chert or flint	7.4–13.0	11.4–12.2	12
Quartzite	7.0–13.2	11.7–14.6	14
Sandstone	4.3–12.1	9.2–13.3	12.5
Marble	2.2–16.0	4.4–7.4	7
Siliceous limestone	3.6–9.7	8.1–11.0	10.5
Granite	1.8–11.9	8.1–10.3	10
Dolerite	4.5–8.5	Average 9.2	9.5
Basalt	4.0–9.7	7.9–10.4	10
Limestone	1.8–11.7	4.3–10.3	9
Glacial gravel	—	9.0–13.7	13
Sintered fly ash (coarse and fine)	—	5.6	7
Sintered fly ash (coarse and natural aggregate fines)	—	8.5–9.5	9

10.2 Measurement of the coefficient of thermal expansion

There is no standard method for measuring the coefficient of thermal expansion for concrete in CEN, ISO or ASTM although a method for repair materials is provided in BS EN 1770. There is a provisional standard written by the American Association of State Highway and Transportation Officials, AASHTO TP 60-00, which is reported to require equipment not freely available.

While in design α_c is assumed to be constant for a particular concrete, in fact it varies with both age and moisture content. Semi-dry concrete has a slightly higher coefficient of thermal expansion than saturated concrete.[17] It is important therefore that testing is undertaken under conditions that reflect the service environment or which are conservative in relation to the value obtained.

In-house methods have to be used. Typically, measuring points would be fixed to a concrete specimen that is placed on roller bearings in a water tank. The specimen is left in the water until there is equilibrium of temperature, and a set of length readings taken. The specimen is then heated to, say, 80°C and kept constant until this temperature is achieved throughout the specimen depth. A second set of readings is taken and the coefficient of thermal expansion calculated.

When testing for early-age values, this may be achieved using a large insulated cube (commonly 1m^3 with 100mm polystyrene insulation on all faces) with embedded thermocouples and strain gauges. Both temperature and strain change are measured and α_c is calculated during cooldown.[6]

The concrete mix proportions for the test should be those that are expected to give the target mean compressive strength.

10.3 Factors influencing the coefficient of thermal expansion

As aggregate comprises about 70% of the concrete volume, this has the dominant effect on the coefficient of thermal expansion as shown in Table 3.

Reducing paste volume will lead to a small reduction in the coefficient of thermal expansion but this change is significantly less than that achieved by changing aggregate type.

11. Thermal conductivity

The thermal conductivity of concrete, λ_c, determines the rate at which heat will be transported through it and hence the rate of heat loss. While it is not required for general design it may be necessary when estimating temperature rise and temperature differentials in some specific situations as follows:

- Predicting early-age temperature rise and differentials due to heat of hydration.
- Estimating temperature differentials under transient conditions, for example, in oil storage vessels that are regularly filled and emptied.

11.1 How thermal conductivity is dealt with in BS EN 1992-1-1

Thermal conductivity is not dealt with in BS EN 1992-1-1.

11.2 Measurement of thermal conductivity

The measurement of thermal conductivity is addressed in BS EN 12667, which provides a general method for testing the thermal performance of building materials. However, as the test involves imposing a temperature gradient through the specimen it is difficult to achieve moisture stability and specimens are generally dried before testing. The values obtained will therefore be more representative of the late-life performance. If values are to be used in early-age analyses, the measured value should be increased by 15%.

11.3 Factors influencing thermal conductivity

There are three principal factors influencing the thermal conductivity of concrete:

1. The aggregate type.
2. The aggregate volume – aggregate has a higher thermal conductivity than both cement and water.
3. The moisture content – as concrete hydrates and dries, the space previously occupied by water empties and the conductivity reduces.

Published values of thermal conductivity vary considerably but are typically within the range 1.0–2.5W/m°C . Values derived for use in thermal analysis at early age and late life are given in Table 4 (from Appendix 2 of CIRIA C660[6]). The lower values at late life reflect increased hydration.

Table 4
Thermal conductivity of concrete using different aggregate types[6] (cement content = 350kg/m^3, w/c = 0.5).

Rock type	Thermal conductivity of concrete (W/m°C)			
	Sand and aggregate from same rock type		Aggregate from defined rock type with siliceous sand	
	Early age	Late life	Early age	Late life
Quartzite and siliceous gravels with high quartz content	2.9	2.5	2.9	2.5
Granite, gabbros, hornfels	1.4	1.2	2.0	1.8
Dolerite, basalt	1.3	1.1	1.9	1.7
Limestone, sandstone, chert	1.0	0.9	1.8	1.6

12. Specific heat

The specific heat of concrete, c_p, is required in the determination of thermal diffusivity, D, (through the expression $D = \lambda_c / c_p \rho$) used in thermal analysis. The range of values for concrete may vary from 0.75 to 1.17kJ/kg°C.[22] This is a very significant variation, indicating that the temperature rise associated with a particular amount of heat generated may vary by as much as ± 20% from a mean value of about 0.96kJ/kg°C. It is particularly important, therefore, that a representative value is used in early-age models for temperature prediction based on heat generation from the cement.

12.1 How specific heat is dealt with in BS EN 1992-1-1

Specific heat is not dealt with in BS EN 1992-1-1.

12.2 Measurement of specific heat

Specific heat is generally measured using calorimetry, although it is evident that it may be predicted with a reasonable degree of accuracy using the method of mixtures and values for the individual constituents.

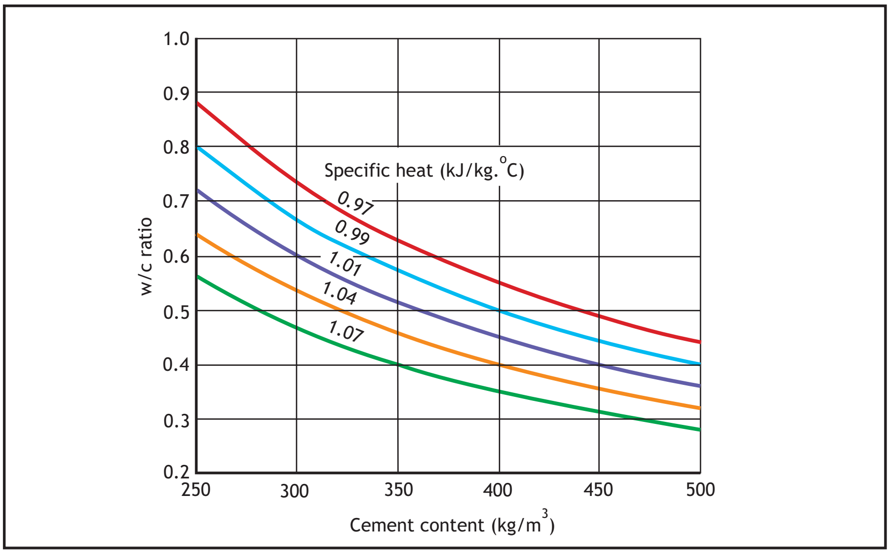
12.3 Factors influencing specific heat

Two factors in particular influence the specific heat of concrete:

1. The aggregate type. Aggregate constitutes the largest proportion of the mass. The specific heat for rocks ranges from 0.8 to 1.0kJ/kg°C and for a typical structure this may result in a 15% difference for concrete.
2. The water content. Water has a specific heat that is four to five times that of the other mix constituents. Dealing with the water content is more complicated as the specific heat differs for free water (4.18kJ/kg°C) and bound water (2.22kJ/kg°C) in concrete.

Therefore to calculate the specific heat for concrete, the relative amounts of free and bound water need to be known and this is determined by the degree of hydration. A method is described in CIRIA C660[6] and values derived from this model for use in early-age thermal analysis are given in Figure 17 for concretes with a range of cement contents and w/c ratios. Late-life values may be 5–10% lower.

Figure 17
The relationship between cement content, w/c ratio and early-age specific heat of concrete (assuming the specific heat of the aggregate is 0.8kJ/kg°C).



13. Fire resistance

Concrete is non-combustible and does not support the spread of flames. It produces no smoke, toxic gases or emissions when exposed to fire and does not contribute to the fire load. Not surprisingly, the European Commission has given concrete the highest possible fire designation, namely A1.

Concrete has a slow rate of heat transfer which makes it an effective fire shield for adjacent compartments, and under typical fire conditions concrete retains most of its strength. Structural fire design is dealt with in BS EN 1992-1-2.

13.1 The effects of fire

The effects of fire on concrete are loss of strength of the matrix and spalling of the concrete surface. Loss of strength of concrete starts as a result of moisture loss and microcracking but the effect is modest up to about 300°C, being of the order of 15%. At temperatures above 300°C the strength loss is much more severe and at 500°C the loss may be approaching 50%.[23] Due to the slow heat transfer through concrete, high temperatures are normally limited to the surface zone and the section retains most of its strength. Spalling can occur with most types of concrete but the severity depends upon the aggregate type, the strength class and the moisture content. Sometimes explosive spalling can be caused by the increase in vapour pressure as water turns into steam. Spalling is more likely in higher-strength concrete as its ability to relieve the vapour pressure reduces. Even when spalling occurs, the integrity of the remaining reinforced concrete is usually adequate.

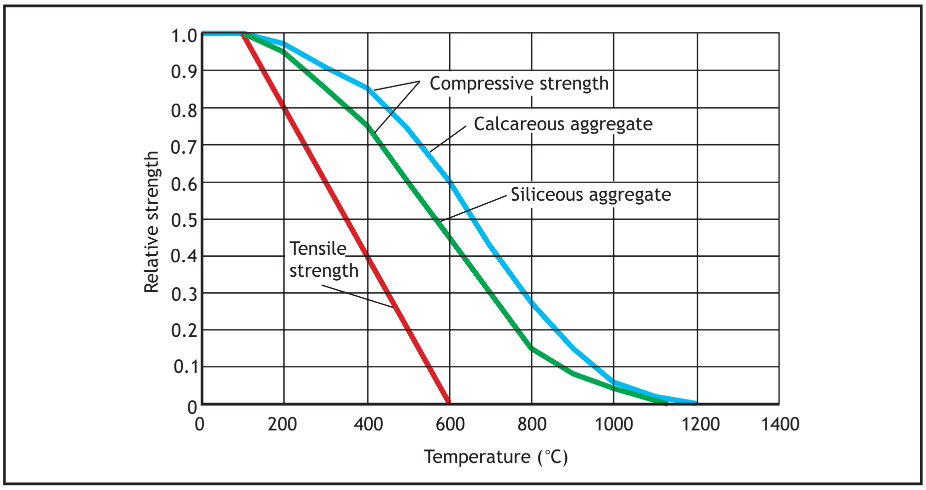
13.2 How fire resistance is dealt with in BS EN 1992-1-2

BS EN 1992-1-2 provides three methods of determining adequate fire resistance. These are: tabulated data (for member analysis only); simplified calculation methods (for member analysis or parts of structures); and advanced calculation methods (for all applications including global structural analysis). In special cases fire engineering methods, where fire levels and resistance are calculated, may be used.

Information on fire resistance is given in BS EN 1992-1-2, Section 3, *Materials* and also in BS EN 1991-1-2: *Fire actions*. In BS EN 1992-1-2 a distinction is made between the performance of concretes using siliceous as opposed to calcareous aggregates, the latter having the better performance at a given temperature as shown in Figure 18.

While BS EN 1992-1-2 states that the tensile strength should normally be (conservatively) ignored, information is provided which may be used in either the simplified or advanced calculation method. This is also shown in Figure 18.

Figure 18
The effect of temperature on strength.

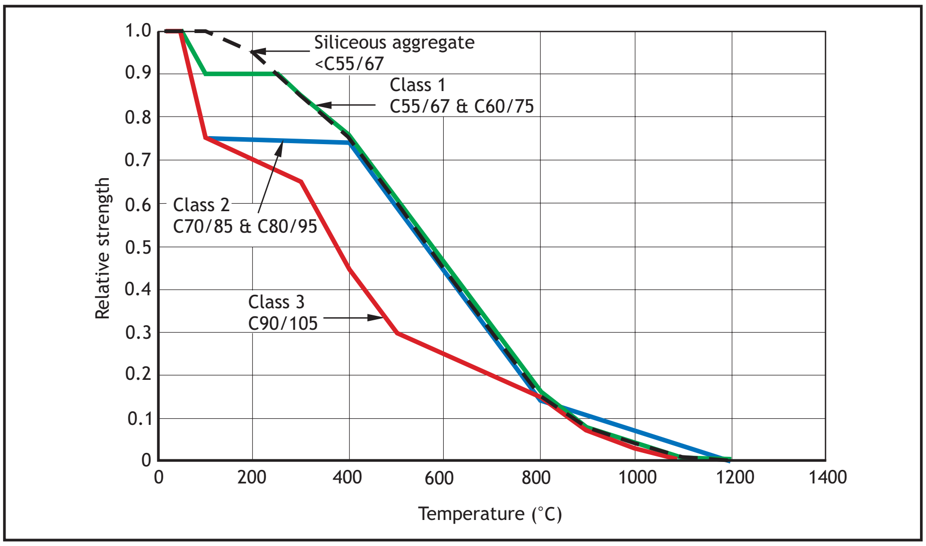


High-strength concrete is dealt with separately in Section 6 of BS EN 1992-1-2 with information presented for three strength classes defined as NDPs in the corresponding UK National Annex as follows:

- Class 1 C55/67 and C60/75
- Class 2 C70/85 and C80/95
- Class 3 C90/105.

The effect of temperature on the compressive strength of these classes is shown in Figure 19. No distinction is made for aggregate type. High-strength concrete is assumed to be more adversely affected by temperature.

Figure 19
The effect of temperature on compressive strength of high-strength concrete.



BS EN 1992-1-2 also provides expressions for estimating the stress–strain relationship at elevated temperature (related to compressive strength) and for thermal properties such as thermal elongation, specific heat and thermal conductivity.

With regard to explosive spalling, this is related to moisture content. The UK National Annex recommends the adoption of the recommended value of 3% (by mass), below which explosive spalling is unlikely.

13.3 How fire resistance is measured

There is a vast database of concrete fire testing over many years, upon which the fire rating of concrete members is based. Historically, laboratory ovens limited the size of structural members which could be tested; however, more recently large-scale tests have been performed at facilities such as the Building Research Establishment facility at Cardington. Criteria for fire testing are based on maintenance of structural integrity and restricting transfer of heat and smoke.

13.4 Improving the fire resistance of concrete

In most normal situations, concrete can be considered to be sufficiently fire resistant, so that further enhancement is not necessary. For a few extreme situations some enhancement of the fire protection or resistance may be required. Some possible approaches are as follows:

- Use of limestone aggregates rather than siliceous aggregates such as flint.
- Use of lightweight aggregate concretes. When dry, performance in fire is very good, but laboratory tests indicate possible poor performance if they are saturated when the fire begins.[23]
- Calcium aluminate cement has a higher resistance to strength loss than other cement types. While this cement is widely used for non-structural applications, for example, refractory linings, there is still debate over its suitability for structural applications and local provisions need to be followed.
- Recognition that high-strength, low-permeability concrete is more prone to spalling. In some situations, structural considerations override that of fire performance, and there may be little practical scope for reducing concrete strength. In such circumstances BS EN 1992-1-2 recommends the option of using not less than 2kg/m^3 of monofilament polypropylene fibres. The mechanism is believed to be the fibres melting and being absorbed in the cement matrix, providing voids for release of high pressure in the pores caused by steam build-up. However, further research is required to confirm the exact mechanism.[23]

Notwithstanding the above recommendations, it should be noted that in design, the normal approach is to improve the fire resistance of an element or structure rather than the concrete itself. The most widely used approach is to increase the cover to reinforcement or to use render and plaster coatings. Where the loss of function can have extreme consequences, sacrificial layers of concrete have been provided containing a stainless steel mesh. In the broader context, fire safety engineering deals not only with passive fire protection but also with the response of a building to fire, taking into account all of the measures introduced through design, the way in which the elements may interact, and measures employed to ensure safety of the occupants and contents.

14. Adiabatic temperature rise

For the control of early-age thermal cracking, a limit may be placed either on the temperature rise of concrete or on allowable temperature differentials within or between elements. Compliance may require either appropriate concrete selection based on historical data, some initial testing of the proposed concrete and/or some full-sized trials or thermal modelling which requires information on the heat generation of the concrete.

Full-scale trials are generally expensive and the contractor may require the concrete producer to undertake some initial testing of the concrete. This may include the adiabatic temperature rise, i.e. the temperature rise under perfectly insulated conditions. In practice this condition is difficult to achieve and the concrete producer may use an approximation of the adiabatic temperature rise.

14.1 The need for adiabatic temperature rise data

CIRIA C660[6] provides temperature rise data for a range of concrete mix types in relation to the cement content and type, and a simple numerical model for estimating the temperature rise and thermal gradients in walls. The model uses adiabatic temperature rise data as a basis for temperature prediction.

14.2 How early-age temperature rise is dealt with in BS EN 1992-3

The control of early-age thermal cracking is dealt with in BS EN 1992-1-3 which covers design of liquid-retaining and containment structures. However, reference is made to BS EN 1992-1-1, Section 7.3 for the design of reinforcement to control cracking. For conditions of conti-nuous edge restraint, i.e. a wall on a rigid foundation, the crack width is determined in part by the magnitude of the restrained contraction. No guidance is given on temperature rise and fall in relation to the concrete but CIRIA C660[6] provides data for a range of concretes and element sizes.

14.3 Measuring the approximate adiabatic temperature rise

There are numerous tests that may be used to measure the heat-generating capacity of a cement type or concrete.

Two European Standard tests are available for the classification of cement. BS EN 196-8 describes the heat of solution method which involves dissolving hydrated cement in a solution of acids and recording the temperature rise in an insulated container. The method adopted by UK cement manufacturers is the semi-adiabatic test to BS EN 196-9. This test is carried out on a mortar sample comprising 360g binder, 1080g sand and 180g water. This scales up to a mix with a binder content of 500kg/m^3 with a w/b ratio of 0.5 and is therefore representative of a high-cement content concrete. The sample is placed in a calorimeter and the temperature rise, typically between 10 and 50°C, is measured and compared with an inert control sample. The temperature rise is converted to heat generated

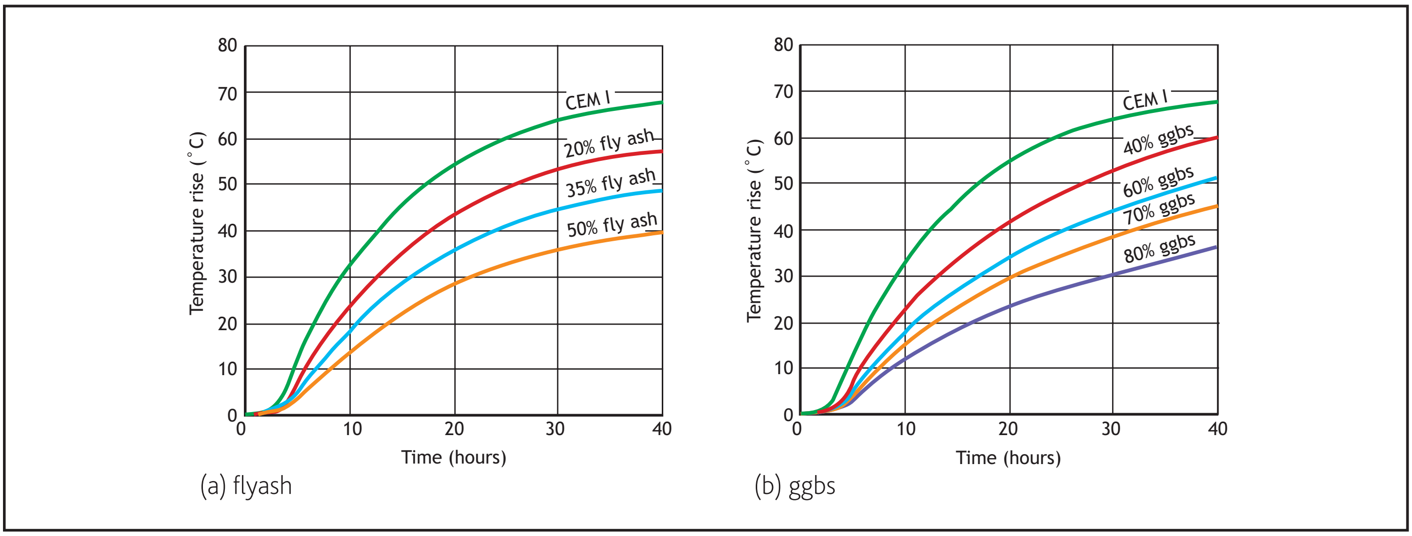


Figure 20
Semi-adiabatic test results for concretes containing (a) fly ash and (b) ggbs[24] (binder content = 500kg/m^3).

per unit weight of cement (kJ/kg) based on the mass and specific heat of the sample and calorimeter. The specific heat of the mortar is about 1.1kJ/kg°C.[24] Some typical results obtained using the semi-adiabatic test are shown in Figure 20 for a range of cement combinations.[25]

When generating data to make predictions of temperature rise, the most reliable approach is to test concrete using constituents and proportions that are the same as, or at least representative of, the mix to be used in practice. While it may be difficult to do this at design stage, it is advisable, where thermal cracking is critical, to undertake testing as soon as the concrete mixes have been established. On very large contracts, trial pours may be required and this provides an opportunity to obtain representative in-situ data. Under conditions where a high temperature rise is expected (i.e. thick sections, concretes with a high cement content, or when placing at high temperature), a hot-box test is appropriate to provide information on temperature rise. This may be achieved by casting a cube, commonly 1m^3, which is insulated on all faces with at least 100mm thick polystyrene.

14.4 Factors influencing the adiabatic temperature rise

As it is the cement that produces heat, the adiabatic temperature rise may be limited by minimising the cement content and using cements containing significant proportions of fly ash or ggbs. Low-heat and very low-heat cements should be considered. If compressive strength controls the mix design, concrete producers typically use admixtures to enable the w/c ratio to be maintained at a lower cement content. Where the minimum cement content is limited by durability and the strength achieved is higher than that required for structural purposes, then mineral additions should be used at the highest level consistent with the strength class and durability requirements. If achieving the required consistence is a problem, the use of filler aggregate might be preferable to increasing the cement content.

If it is necessary to reduce the temperature rise in-situ then a number of measures are available[6] as follows:

- Cooling the aggregates by spraying with water or liquid nitrogen.
- Using ice to partially replace mix water.
- Using liquid nitrogen to cool the mixed concrete.
- Using cooling pipes in the element to remove heat as it is generated.
- Using low-insulation formwork to permit rapid heat loss (in thin sections when temperature gradients are not critical).

In addition to reducing early-age temperature rise, the following measures may be used to reduce the risk of thermal cracking:[6]

- Using aggregate with a low coefficient of thermal expansion (for example, limestone).
- Using aggregate which leads to a high tensile strain capacity (for example, limestone).
- Reducing restraint by planning pour sizes and sequence of construction.
- Reducing restraint by introducing full- or partial-movement joints.
- Using high-insulation formwork or surface insulation to reduce heat loss in thick sections when temperature gradients are critical.

15. Durability

Design for durability uses a deemed-to-satisfy approach with limits on strength class, mix proportions and cover provided for a variety of exposure conditions. In some cases the constituent materials are also specified or their properties defined. Durability recommendations for the UK are provided in BS 8500 Part 1 which is the complementary British Standard to BS EN 206-1, which is in turn referenced in BS EN 1992-1-1, Section 4. BS 8500-1 gives recommendations for an **intended working life** of at least either 50 or 100 years. However the UK National Annex for Eurocode 0 recommends an **indicative design working life** of 120 years for Category 5 structures which includes bridges and civil engineering structures. It can be assumed that the BS 8500 provisions for 100 years are also suitable for an indicative design working life of 120 years.

15.1 Preventing reinforcement corrosion

Corrosion of reinforcement occurs when the protection normally afforded by the alkaline environment in concrete is lost, either as a result of carbonation or the ingress of chlorides. Exposure conditions are categorised as follows:

- X0 – no risk of reinforcement corrosion or attack
- XC – reinforcement corrosion induced by carbonation
- XD – reinforcement corrosion induced by chlorides other than seawater
- XS – reinforcement corrosion induced by chlorides from seawater.

For each of these environmental actions and levels of severity, recommended limiting values are given for maximum w/c ratio, minimum strength class and minimum cement content. BS 8500-1 provides limits specific to the UK and differs from BS EN 206-1 in that it allows a trade-off between concrete quality and cover.

15.2 Preventing freeze–thaw damage

BS EN 8500-1 gives four levels of freeze–thaw exposure, namely XF1 to XF4. Air entrainment is the recognised means for resisting freeze–thaw attack. Minimum air contents are required, typically in the range 3–5.5%, with the higher levels required in concretes having a smaller maximum aggregate size. Also limits are placed on maximum w/c ratios and minimum strength class.

In addition, for each exposure class, options are provided in BS 8500-1 for concrete with and without air entrainment, the latter having a higher strength class. However, it is stated that the air-entrained option will provide superior freeze–thaw resistance and consideration should be given to this option, particularly for pavements and hardstandings. Practical difficulty in achieving air entrainment in concrete with a strength class of C35/45 or higher is also noted.

15.3 Preventing chemical attack

BS EN 206-1 defines only three levels of chemical attack from natural soil and static groundwater (XA1 to XA3). The XA action is redefined in BS 8500-1, as for some of the exposure classes limiting values and test methods recommended by BS EN 206-1, differ significantly from UK practice. BRE Special Digest 1[26] covers a wider range of environmental actions including those for mobile groundwater, acids and brownfield sites, and BS 8500-1 recommends this BRE approach. Recommendations in BS 8500-1 are therefore based on Design Chemical (DC) classes which take account of the sulfate level, the nature and level of acidity, the mobility of the groundwater and the hydrostatic head. BS 8500-1 also provides guidance on additional protective measures (APM) that may be employed.

15.4 Avoiding alkali–silica reaction

BS EN 206-1 deals with alkali–silica reaction through a general requirement that the constituent materials shall not contain harmful ingredients in such quantities that may be detrimental to durability. BS 8500-2 requires the concrete producer to take action to minimise the risk of potentially damaging ASR. Following the guidance in BRE Digest 330[27] is deemed to satisfy this requirement.

15.5 Performance-based design for durability

In addition to the deemed-to-satisfy prescriptive approach, BS EN 206-1 also permits performance-based design methods for durability. Such an approach is considered to be appropriate under a range of circumstances including when:

- the working life significantly exceeds 50 years
- the structure is 'special', requiring a low probability of failure
- the environmental actions are particularly aggressive.

In adopting the performance-based approach based on testing, it is required that tests are proven and representative of actual conditions, and have approved performance criteria. When analytical methods are used these should be calibrated against test data that are representative of actual conditions.

16. The use of recycled aggregates

The UK aggregate industry optimises the use of recycled aggregate (RA) in support of the wider aim of sustainable development.

16.1 Requirements of BS EN 206-1 and BS 8500

BS EN 206-1 notes that it does not include provisions for recycled aggregates but that suitability may be established on the basis of the general requirements, and either a European Technical Approval, or national standards or provisions that refer specifically to their use in concrete conforming to BS EN 206-1.

BS 8500-2 permits the use of RA and RCA (recycled concrete aggregate) which conform to specific requirements of BS 8500 and which, when used in combination with natural aggregates, meet the requirements of BS EN 12620. The use of RCA for coarse aggregate is limited to strength classes ≤ C40/50 and to less aggressive exposure classes including carbonation and the lowest levels of freeze–thaw and choride exposure. BS 8500-2 contains a full specification for RCA but not for RA as there are insufficient data to provide a robust general specification for every possible type of RA.

16.2 Properties influenced by the use of Recycled Aggregate and Recycled Concrete Aggregate

A particular feature of both RA and RCA is a lower specific gravity than most primary aggregates. Oven-dried values may typically be 2.0–2.4[30,31] compared with values more typically in the range 2.3–2.8 for primary aggregates. Lower specific gravity indicates higher absorption and less stiffness. Properties of concrete that are influenced by these factors should therefore be considered when RA or RCA are used, in particular elastic modulus, creep and shrinkage, which are all influenced by the aggregate stiffness. Where deflections and creep deformations are of importance for a contract, the use of RA should be considered carefully. It should also be appreciated that RA and RCA are currently used in combination with primary aggregate and any effects will therefore be diluted in relation to the relative proportions of the materials used.

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  - Part 1-2 – *General rules – Structural fire design (including NA)*. BSI, 2004.
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- ISO/WD 1920-9, *Testing concrete – Part Y: Determination of creep of concrete cylinders in compression* (draft).

Appendix A

Table A1
Extract from BS EN 1992-1-1 - Table 3.1.

Strength classes for concrete

	C12/16	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75	C70/85	C80/95	C90/105	Analytical relation
f_{ck} Characteristic cylinder strength (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck, cube}$ Characteristic cube strength (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} Target mean cylinder strength (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck} + 8$ (MPa)
f_{ctm} Mean axial tensile strength (MPa)	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.8	5.0	$f_{ctm} = 0.30 \times f_{ck}^{(2/3)} \leq$ C50/60 $f_{ctm} = 2.12 \ln[1 + (f_{cm}/10)]$ > C50/60
$f_{ctk, 0.05}$ Characteristic axial tensile strength, 5% fractile (MPa)	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9	3.0	3.1	3.2	3.4	3.5	$f_{ctk, 0.05} = 0.7 \times f_{ctm}$ 5% fractile
$f_{ctk, 0.95}$ Characteristic axial tensile strength, 95% fractile (MPa)	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3	5.5	5.7	6.0	6.3	6.6	$f_{ctk, 0.95} = 1.3 \times f_{ctm}$ 95% fractile
E_{cm} Mean secant modulus of elasticity (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0.3}$ (f_{cm} in MPa)

Table A2
Nomenclature and composition for cements and combination types[a].

Broad designation[b]	Composition	Cement/combination types (BS 8500)
CEM I	Portland cement	CEM I
SRPC	Sulfate-resisting Portland cement	SRPC
IIA	Portland cement with 6–20% fly ash, ground granulated blastfurnace slag, limestone, or 6–10% silica fume[c]	CEM II/A-L, CEM II/A-LL, CIIA-L, CIIA-LL, CEM II/A-S, CIIA-S, CEM II/A-V, CIIA-V, CEM II/A-D
IIB-S	Portland cement with 21–35% ground granulated blastfurnace slag	CEM II/B-S, CIIB-S
IIB-V	Portland cement with 25–35% fly ash	CEM II/B-V, CIIB-V
IIB+SR	Portland cement with 25–35% fly ash	CEM II/B-V+SR, CIIB-V+SR
IIIA[d, e]	Portland cement with 36–65% ground granulated blastfurnace slag	CEM III/A, CIIIA
IIA+SR[e]	Portland cement with 36–65% ground granulated blastfurnace slag with additional requirements that enhance sulfate resistance	CEM III/A+SR[f], CIII/A+SR[f], CIIIA+SR
IIIB[e, g]	Portland cement with 66–80% ground granulated blastfurnace slag	CEM III/B, CIIIB
IIIB+SR[e]	Portland cement with 66–80% ground granulated blastfurnace slag with additional requirements that enhance sulfate resistance	CEM III/B+SR[f], CIIIB+SR[f]
IVB-V	Portland cement with 36–55% fly ash	CEM IV/B(V), CIVB

Key

a There are a number of cements and combinations not listed in this table that may be specified for certain specialist applications. See BRE *Special Digest 1*[2] for the sulfate-resisting characteristics of other cements and combinations.

b The use of these broad designations is sufficient for most applications. Where a more limited range of cement or combinations types is required, select from the notations given in BS 8500–2: 2006, Table 1.

c When IIA or IIA–D is specified, CEM I and silica fume may be combined in the concrete mixer using the *k*-value concept; see BS EN 206–1: 2000, Cl. 5.2.5.2.3.

d Where IIIA is specified, IIIA+SR may be used.

e Inclusive of low early strength option (see BS EN 197–4 and the 'L' classes in BS 8500–2: 2006, Table A.1).

f '+SR' indicates additional restrictions related to sulfate resistance. See BS 8500–2: 2006, Table 1, footnote D.

g Where IIIB is specified, IIIB+SR may be used.

Table A3
Standard methods for measurement of physical properties.

Property	Standard tests	Comments
Compressive strength, cylinders and cubes	BS EN 12390-3	
Tensile splitting strength	BS EN 12390-6	
Flexural strength	BS EN 12390-5	
Direct tensile strength	No standards	
Bond strength	BS EN 10080	Bending test has replaced the direct pull-out test
Static modulus of electricity (secant modulus)	Standard BS EN test under development	
Tensile strain capacity	No standards	May be estimated from mean tensile strength divided by mean elastic modulus
Dynamic modulus of elasticity (≈ initial tangent modulus)	BS EN 12504-4 BS 1881-209	This BS EN gives the procedure for determination of ultrasonic pulse velocity (UPV). The procedure for the conversion of UPV into an initial tangent modulus is covered in BS 1881-203. It is expected that this procedure will be included in the UK National Annex to BS EN 12504-4 *Procedure for measuring the dynamic modulus of elasticity* (≈ initial tangent modulus)
Creep	No standard EN test. ASTM C 512-02 ISO/WD 1920-Y	Measures total creep + drying creep Measure of creep in compression
Autogenous shrinkage	No standards	
Drying shrinkage of concrete	No standard EN test. ASTM C 157/C ISO/WD 1920-X	
Drying shrinkage of aggregate	BS EN 1367-4	Though measured in concrete, this does not measure the basic (unrestrained by reinforcement) drying shrinkage strain of concrete
Autogenous shrinkage	No standards	
Coefficient of thermal expansion	No standards	
Thermal conductivity	BS EN 12667	
Specific heat	No standards	
Adiabatic heat	BS EN 196-9	

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Properties of Concrete for use in Eurocode 2

This publication is aimed at providing both civil and structural design engineers with a greater knowledge of concrete behaviour. This will enable the optimal use of the material aspects of concrete to be utilised in design. Guidance relates to the use of concrete properties for design to Eurocode 2 and the corresponding UK National Annex.

In the design of concrete structures, engineers have the flexibility to specify particular concrete type(s) to meet the specific performance requirements for the project. For instance where calculated deflections exceed serviceability limits, the designer can achieve lower deflections by increasing the class of concrete and the associated modulus of elasticity, rather than by resizing members. This publication will assist in designing concrete structures taylor-made for particular applications.

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