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Guidance to the engineering properties of concrete

November 2006

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ABBREVIATIONS

c_v	Coefficient of variation
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_{ck}	Specified characteristic cylinder compressive strength
$f_{ck.cu}$	Specified characteristic cube compressive strength
f_{cm}	Target mean cylinder compressive strength
f_{ct}	Axial tensile strength
f_{ctd}	Design tensile strength
f_{ctm}	Mean axial tensile strength
$f_{ctm,fl}$	Mean flexural strength
$f_{ct,sp}$	Tensile splitting strength

$\varepsilon_{cc}\ (\infty,t_0)$	Creep deformation at time t = ∞
$\varepsilon_{ca}(t)$	Autogenous shrinkage strain up to time t
$\varepsilon_{ca}(\infty)$	Autogenous shrinkage strain at time t = ∞
$\varphi\ (\infty,t_0)$	Creep coefficient at time t = ∞
σ_c	Constant compressive stress applied at time $t = t_0$

γ_{cE}	Partial safety factor for the E-value of concrete

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1. INTRODUCTION

In the design of concrete structures, engineers use a number of mechanical properties of concrete that are not normally part of the concrete specification. Assumed (safe) values are used and these are often related to the compressive strength used for the design. Section 3 of Eurocode 2, EN 1992-1-1 covers these engineering properties in a unified way. Generally, these mechanical properties are assumed to depend upon the mean concrete compressive strength, which is taken to be the specified characteristic strength plus 8MPa. Values for the properties are calculated from the mean compressive strength. In this respect (the calculation of values based on the specified characteristic strength) the Code does not differ from previous approaches to engineering properties.
The code also outlines in each case how the value of the properties at different ages can be calculated.

Table 1 reproduces the principal strength and deformation characteristics of concrete, as set out in table 3.1 of the Code. Figure 1 shows the principal graphical relationships between on the one hand, specified characteristic compressive strength, and, on the other, target mean compressive strength, mean axial tensile strength and mean (secant) modulus of elasticity.

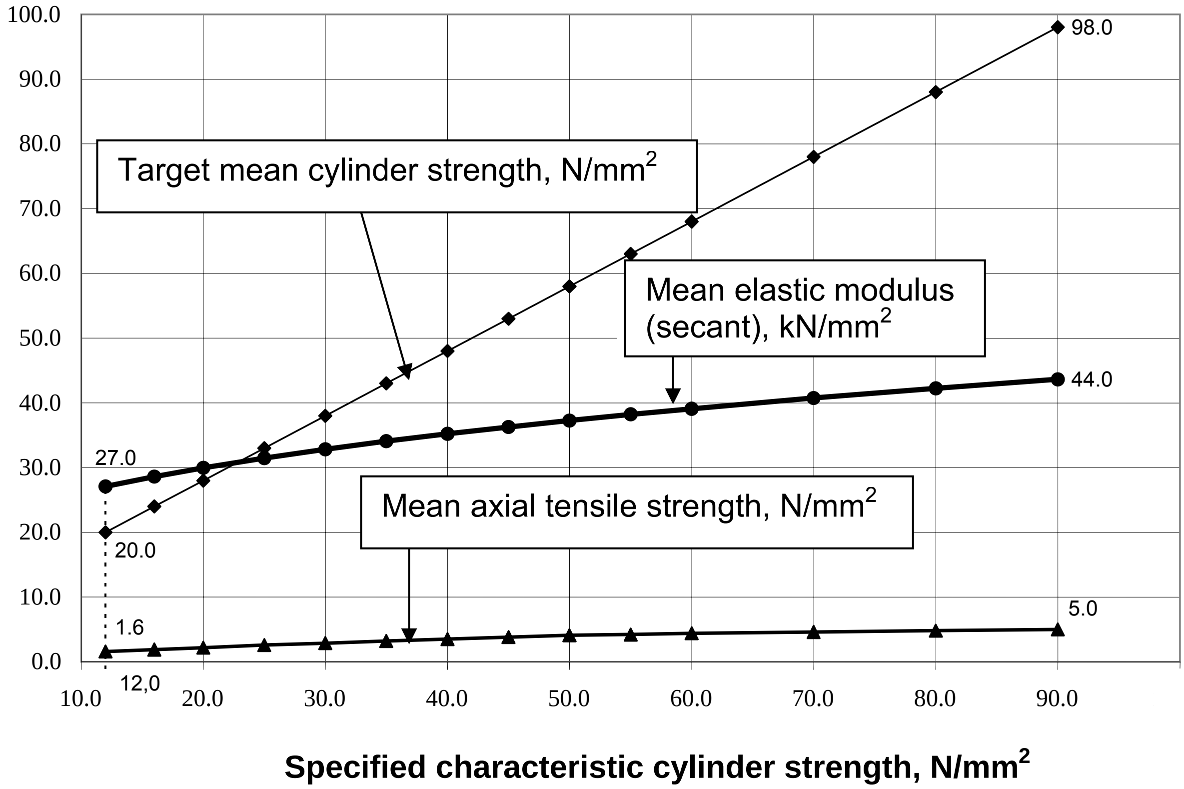


Figure 1: relationships between specified strength, target mean strength, mean axial tensile strength and elastic modulus

Strength classes for concrete															Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (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 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (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
$f_{ctk,0,95}$ (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} (GPa)	27	29	30	31	32	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)

TABLE 1.

Extract from EN1992-1-1, table 3.1

f_{ck}: characteristic cylinder strength
$f_{ck.cu}$: characteristic cube strength
f_{cm}: target mean cylinder strength
f_{ctm}: mean axial tensile strength
$f_{ctk,\,0,05}$: characteristic axial tensile strength, 5% fractile
$f_{ctk,\,0,95}$: characteristic axial tensile strength, 95% fractile
E_{cm}: mean secant modulus of elasticity

These mechanical properties of concrete are used in serviceability calculations. Given current trends to use higher strength concrete, serviceability considerations may start to control the design process [1], and designers may start to look at the design values more closely to identify where economies may be possible.

In most cases, use of the design values given in EN 1992-1-1 is adequate. However, engineers who wish to take advantage of the full potential of concrete construction may find that the assumed design value limits the design possibilities.
For example, in slab design, deflexion is usually the factor limiting the span, and this in turn is linked to the mean modulus of elasticity. If a higher value of modulus could be justified in the design, spans could be increased without increasing slab thickness. For good technical reasons, therefore, designers may wish to specify a higher than assumed value for a property, or seek advice from the producer on how this property can be enhanced. The ability of the concrete industry to satisfy these demands may determine if the structure is built in concrete or some other material.

This publication covers:

- background to why and when the property is important to designers;
- how the property is determined in EN 1992-1-1;
- how the property can be measured;
- how the measured value may be used in design;
- options for enhancing the value of the property.

2. A NOTE ON SOME ASSUMPTIONS UNDERLYING THE CODE

2.1 Importantly, Eurocode 2 assumes that design and construction will :

- be subject to quality control procedures;
- be undertaken by appropriately qualified and experienced personnel with adequate supervision;
- use materials and products as specified;
- meet the requirements for execution and workmanship given in ENV 13670, 'Execution of concrete structures'.

2.2 EN 1990, Basis of 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 e.g. through flexural failure, shear failure, buckling, failure of anchorages, etc.
- **Serviceability Limit States** (SLS) correspond to conditions beyond which specified service requirements are no longer met e.g. excessive deformation, excessive cracking or stress.

In design, both these 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.
50 years is often used as the design working life of building and other common structures.

3. RELEVANT CONCRETE ENGINEERING PROPERTIES

The only engineering property of concrete that is routinely specified is the characteristic compressive strength, expressed in EN 206-1 as a compressive strength class. It is used as an input in all concrete design procedures. In most design, the characteristic cylinder strength obtained from the specified compressive strength class is *also* used to determine design values for a number of other engineering properties including tensile strength, E-value and creep. Concrete producers are familiar with supplying concrete with a specified compressive strength class and demonstrating conformity. Therefore this publication will not cover these aspects of compressive strength.

The following additional engineering properties of concrete are used in some design procedures:

- tensile strength;
- modulus of elasticity;
- creep;
- shrinkage;
- coefficient of thermal expansion;
- fire resistance.

Guidance on these properties is given in the following sections.

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 (stretches). EN 1992-1-1 gives provisions for using reinforcement with different ductility. The use of fibres will improve the ductility of concrete, but this is outside the scope of this publication and EN 1992-1-1.

4. COMPRESSIVE STRENGTH

4.1 Background

Eurocode 2 uses the **characteristic compressive strength** of concrete as the basis of design calculations. This characteristic strength has a statistical basis: it is that strength below which 5% of results may be expected to fall. Although results below the specified minimum strength (i.e. specified characteristic strength minus 4N/mm^2) may need to be investigated, especially in critical locations, they do not mean that the element or structure is necessarily unsafe or unserviceable.

The **design compressive strength** of concrete, f_{cd}, used in design to Eurocode 2 is taken as:

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

where,

f_{ck} = characteristic cylinder compressive strength of concrete at 28 days
γ_c = partial (safety) factor for concrete
α_{cc} = a coefficient to allow for long term and loading effects.

In the Eurocode, the **recommended value** for the partial safety factor is 1.5, and that for the α_{cc} coefficient 1.00. However, these are subject to each country's National Annex to Eurocode 2. For instance, in UK, the value for the α_{cc} coefficient is generally 0.85.
The consequence of low results is part of what is covered by the concrete partial safety factor.

Compressive strength of concrete is not one unique property. For test specimens, compressive strength is a function of the aspect ratio (length / width) and the rate of loading. In standard testing, aspect ratios are standardised and the range of the rates of loading limited, so that the rate does not have a significant effect on the result. With short specimens, e.g. cubes, the test machine platens restrain the lateral expansion of the specimen while it is being loaded, and this results in a higher value of compressive strength than that obtained from 2:1 cylinders, where there is little lateral restraint in the central section of the cylinder. In EN 206-1 the 2:1 cylinder strength is taken to be about 20% less than the cube strength, but as concrete strength increases this difference becomes smaller, so that cube strength is closer to cylinder strength.

The actual **compressive strength in the structure** is deemed to be less than that obtained by testing standard 2:1 cylinders. The Eurocode and the standard, EN 13791, "Assessment of concrete strength in structures' both apply a factor of 0.85.

Eurocode 2 uses the **cylinder strength** as the basis of design calculations, because in some situations this is close to the failure load of the concrete. For example, in a simple beam without shear links or top steel, the failure load of the concrete is similar to the cylinder strength, when the difference in strength between concrete in test specimens and the structure is taken into account. On the other hand, in concrete elements where the concrete is confined with reinforcement, e.g. a column with shear links, the compressive stress the concrete can withstand before a failure in compression is significantly higher. The Eurocode takes this into account in the design equations.

For obvious reasons, the compressive strength of concrete is a principal criterion in the design of columns. It is fundamental to determining the amount of compression reinforcement required. But it is also important in flexural members such as beams and slabs, where it is the main criterion for determining the stress distribution in a section and, consequently, the amount of tension reinforcement required there. Compression reinforcement is sometimes needed in beams, too.
The compressive strength of concrete is also used to determine shear capacities and shear reinforcement, cracking and crack control reinforcement, deformation etc.

4.2 Estimation of mean compressive strength

Designers use the mechanical properties described in this publication in the checks on the serviceability limit state. Consequently for each property a value appropriate to the **mean** compressive strength is (almost always) used and **not** a value related to the characteristic strength. For simplicity, the mean strength is assumed to be the characteristic strength plus 8 MPa (cylinder), equivalent to 10 MPa 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 8 (10) MPa is usually adequate and there is no justification for using a lower margin.

However, there are situations where the use of a design value linked to a higher mean strength is appropriate. These situations include:

- 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 strength will continue to develop significantly above the 28-day standard strength;
- where the mix design and mean strength depend on a factor other than specified characteristic strength, e.g. the maximum w/c ratio.

4.3 Development of compressive strength with time

EN 1992-1-1 sub-clause 3.1.2(6) gives an equation for the development of compressive strength of concrete at 20°C. The compressive strengths obtained from this equation (dependent on cement strength class) are shown in Figure 2. However, 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 II/B-V, 32,5 or CEM IIIA, 32,5 cements, provided there is water for continued hydration. (Cement types defined in EN 197-1.)

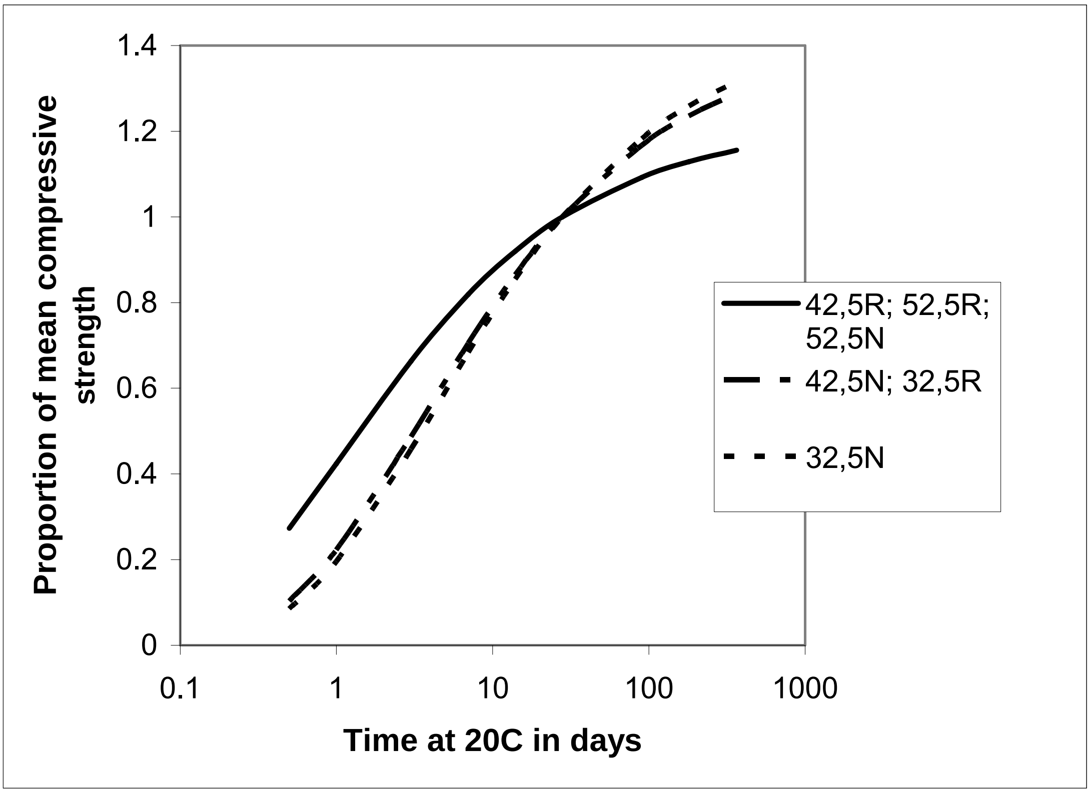


Figure 2: Rate of strength development at 20°C for different cement strength classes

If the designer has information that show the concrete to be supplied will gain strength more rapidly, this information may be used, for example, to reduce the stress/strength ratio in the creep calculation.

EN 1992 does not cover the rate of strength development *in the structure itself* and this will depend upon [3]:

- type of concrete (mainly cement type and content);
- concrete placing temperature;
- ambient temperature;
- section thickness;
- type of formwork.

If the rate of strength development in a section has to be predicted, computer modelling and maturity calculations are necessary. The producer may be required to supply some basic information, which will depend upon the model being used [4], for example, cement type, class and content, and the adiabatic temperature rise curve. 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.

5. TENSILE STRENGTH

5.1 Background

In the design of concrete, tensile strength is used for:

- generally, 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;
- drawing 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 unreinforced concrete sections, e.g. concrete pavements. In this case the designer wants a high tensile strength.
- the design of fibre-reinforced concrete.

It should be noted that high tensile strength can create its own problems. In the case of early thermal contraction, for instance, high tensile strengths lead to increased restraint and shrinkage, and wider crack spacing and wider cracks. To prevent this, additional crack control steel is required.

There are three 'types' of tensile strength: flexural strength (standard test on prisms), tensile splitting strength (standard test on cylinders) and axial tensile strength (no standard test).
For a given concrete, it is important to appreciate that there cannot be a single value for these three types of tensile strength, because the test value varies with:

- the test method;
- the rate of application of load;
- specimen size.

This difference in the values is explained by the 'weakest link' concept. This supposes a tensile failure starting at the weakest link, and once started, spreading rapidly throughout the cross-section. Therefore with a larger area in tension, it is likely that the strength of the 'weakest link' will be lower than with a smaller area, and consequently the measured tensile strength will also be lower.
The practical implication of this is that the values of tensile strength measured using a flexural test will be significantly higher than those obtained by the cylinder splitting test, which in turn will be higher than those obtained using a direct uniaxial tension test – see figures 3 a-c.

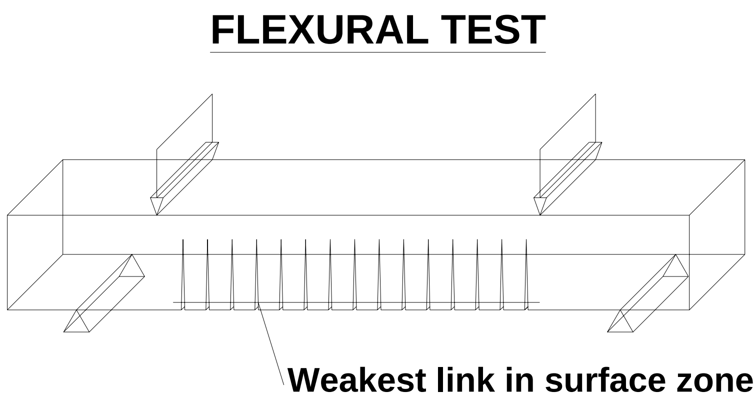


Figure 3a: location of the weakest link in the flexural test

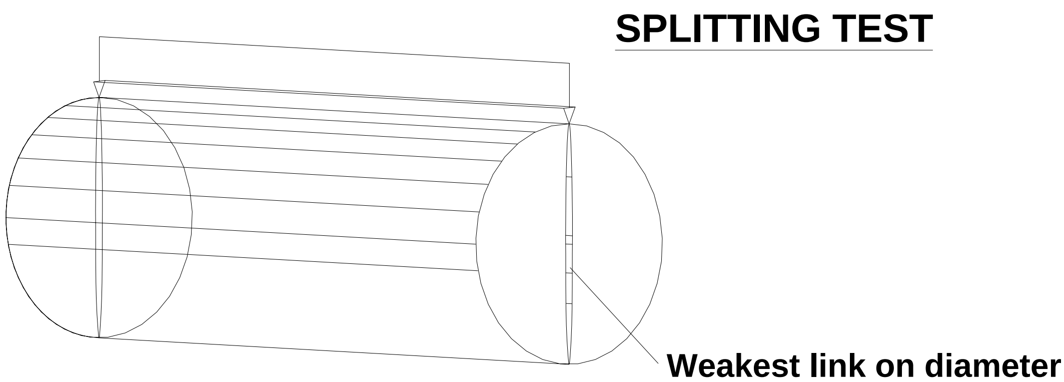
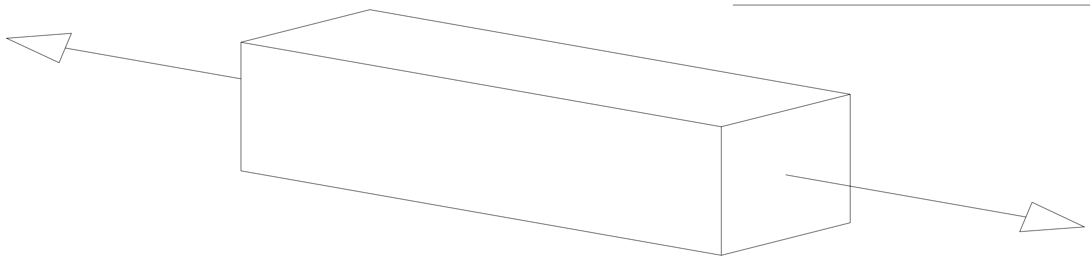


Figure 3b: location of the weakest link in the splitting test

DIRECT TENSILE TEST



Weakest link anywhere in specimen

Figure 3c: location of the weakest link in the direct tensile test

5.2 How tensile strength is dealt with in EN 1992-1-1

In EN 1992-1-1, the term 'tensile strength' refers to the highest stress reached under concentric tensile loading, i.e. the **axial tensile strength** – fig. 3c**.**

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

for compressive strength classes ≤ C50/60

$$f_{ctm} = 0{,}30 \times f_{ck}^{(2/3)} \text{ MPa}$$

and for compressive strength classes > C50/60,

$$f_{ctm} = 2{,}12 \times \log_e (1 + ((f_{ck} + 8)/10)) \text{ MPa}$$

Note 1. (f_{ck} + 8) is the assumed **mean** cylinder strength. This is used in **serviceability** calculations.

Note 2: These values assume that the requirement for strength is controlling the mix proportions.

Unfortunately, this (the mean axial strength) is the definition of tensile strength with which concrete producers are least familiar. However, where the tensile strength is determined by the **tensile splitting test** in accordance with EN 12390-6, EN 1992-1-1 permits the axial tensile strength to be **calculated**, from the tensile splitting strength, and it is taken as 90% of the tensile splitting strength:

$$f_{ct} = 0{,}90 f_{ct,sp}$$

When using this approach, the tensile splitting test should be based on testing concrete that has proportions aimed at giving the target mean strength, as this will result in an estimate of the mean tensile strength.

The **flexural tensile strength** can be measured using the EN 12390-5 test procedure, but in the code, it can also be calculated from the mean axial tensile strength by the following equations:

The flexural strength is the higher of:

a) $f_{ctm,fl} = (1{,}6 - h/1000)\, f_{ctm}$, where h is the total member depth in mm.

or,

b) $f_{ctm,fl} = f_{ctm}$

The first equation indicates that the flexural strength measured on a 100 × 100mm prism in accordance with EN 12390-5 is 1,5 times higher than the axial tensile strength.

As the tensile strength influences early-age thermal cracking, EN 1992-1-1 provides equations for calculating tensile strength at different maturities, but recommends that where the development of tensile strength with time is important, tests should be carried out, taking into account the exposure conditions and the dimensions of the structural member. For practical reasons, the test itself cannot reflect the exposure and dimensions of the structural member; but using the concept of maturity, it is possible to calculate the maturity of the structure at which cracking is expected and to test specimens with this maturity.

5.3 Measuring tensile strength

Where the tensile splitting strength is required, it should be determined in accordance with EN 12390-6. Where the flexural strength is required, it should be determined in accordance with EN 12390-5 using the 4 point method. The alternative method of loading (centre-point loading) has been found to give results 13 per cent higher than the reference method. There is no European or International standard for the measurement of the axial tensile strength. Neither EN 12390-5 nor EN 12390-6 includes information on the precision of the test.

Tensile strength testing produces variable results. Sherriff [5] has shown that the coefficient of variation due to testing of the tensile splitting test was over twice that for cube testing (6.5% compared with 3.2%). In the code, too, a high coefficient is assumed.
Using a combination of probability theory and computer simulation, Sherriff also showed that in order to achieve a reasonable chance of conformity, the producer's design margin must be set high enough to give a failure rate of appreciably less than 1%.
Ryle [6] showed that under laboratory conditions it was possible to get relatively low testing errors with the tensile splitting test, but normal compressive testing machines that were in calibration for cube testing may nevertheless give unreliable results. He concluded by saying that the tensile splitting test was unsuitable as a conformity test for concrete.

5.4 Some practical advice

Because of high test variability, most ready mixed concrete producers resist entering contracts to supply concrete on the basis of a specified characteristic tensile splitting or characteristic flexural strength. On request, producers will supply information on the tensile strength obtained on concrete that has the target mean compressive strength via the initial testing route in EN 206-1. It is recommended that at least three (ideally six) tensile strength specimens are made from each of three batches, one aimed at achieving the target mean strength and one each above and below the target mean strength and then interpolate the results to give the tensile strength at the higher of (f_{ck} + 8) MPa (cylinder) or the target mean strength.

In EN 1992-1-1, sub-clause 3.1.2(9) the **rate of development of tensile strength** is assumed to be the same as that of compressive strength up to the age of 27 days. At 28-days and older, the tensile strength uses a rate coefficient that is to the power of two-thirds that used for compressive strength. However, when knowledge of the development of tensile strength with time is important, the standard recommends testing.

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 producers should try to agree on either the tensile splitting test (EN 12390-6), or the flexural test using the EN 12390-5 reference method. Due to testing variability, at least three (ideally six) specimens should be tested at each age. As neither of these test methods has precision data, all that can be usefully done with the results is to calculate the mean value. The test results relate to the development of tensile strength in standard test specimens and not the structure. Computer modelling and maturity calculations are needed to transpose these data to the expected tensile strength in the structure.

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

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

5.5 Enhancing tensile strength.

Some general approaches:

- increasing the compressive strength will increase the tensile strength;
- the relative volumes of paste and aggregate have little effect on tensile strength [7];
- coarse aggregate type has a significant influence on tensile strength. Concrete containing high quality crushed rock coarse aggregate tends to have higher tensile strength than concrete made with gravels.

Steel fibres do not change the tensile strength of concrete itself, but in concrete elements they control cracking and help avoid catastrophic failure.
Polymer fibres only help to control cracking of concrete in the plastic state

6. MODULUS OF ELASTICITY

6.1 Background

The modulus of elasticity (E-value) of concrete is an important property in design:

- used in the calculation of deflection, often the controlling parameter in slab design;
- used in calculations involving pre- or post- tensioned elements;
- also affects column shortening under load and the stresses due to restrained movements.

The modulus can be measured using either static or dynamic tests, and the two procedures do not give the same measured value of the 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 vibration of a concrete specimen. Producers are more likely to be familiar with tests to determine the static modulus.
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 and the initial tangent modulus, see Figure 4.

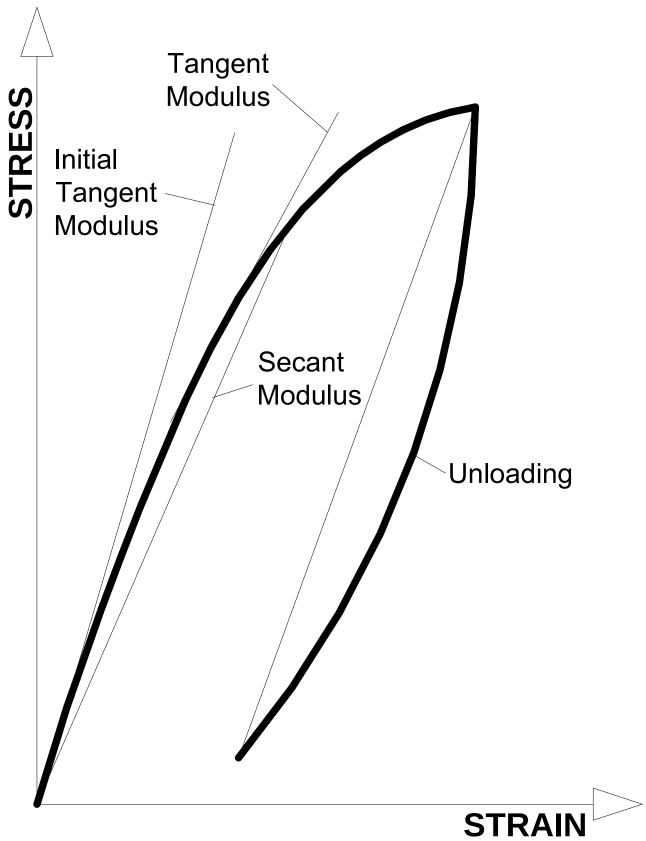


Figure 4: diagrammatic stress – strain relationships for concrete *(after A.M.Neville, 'Properties of Concrete')*

These are measurements of the static modulus. The initial tangent modulus, however, is approximately equal to the dynamic modulus. It can be seen that the tangent moduli only apply to the adjacent part of the stress/strain curve. The most generally useful measure is the secant modulus, and in EN 1992-1-1, it is the **secant modulus**, E_{cm}, that is used in design.
In design, the secant modulus is calculated, or assumed, from the characteristic compressive strength, and modified by two factors:

- reduced by a safety factor, γ_{cE}, to give a value for the modulus, E_{cd} to be used in design. The recommended value of γ_{cE} is 1,2.
- since the E value is in reality modified by creep, the code applies another reduction to the E-value – the creep co-efficient, which typically has a value between 2 to 3.

The E-value modified in these two ways is called the **effective modulus of elasticity.** This is given the notation $E_{c,\ eff}$. and is what is used in design to resist permanent and long-term loading.
For short term loading of concrete elements, the design value of modulus, E_{cd}, is used to determine the movement.

6.2 How E-value is dealt with in EN 1992-1-1
If information is not available on the concrete that will be used, designers have to use the lowest common European value for E-value. For quartzite aggregates, the 'normal' static modulus (the secant modulus) is calculated from the specified characteristic cylinder strength from the equation:

$$E_{cm} = 22[(f_{ck} + 8)/10]^{0,3} \text{ GPa} \qquad (f_{ck} \text{ in MPa})$$

The text of EN 1992-1-1, 3.1.3(2) does not specifically say so, but the equation for quartzite aggregates may also be applied to concretes made with siliceous aggregates.
For limestone and sandstone aggregates, the value is reduced by 10% and 30% respectively and for basalt aggregates, it is increased by 20%. This approach assumes that the designer knows the aggregate to be used; usually, this is not the case until the supplier is selected. Only in the case of very high strength concrete is the type of coarse aggregate usually known and often, specified.

6.3 Measuring the E-value
6.3.1 Test methods
Work is in progress within CEN to develop a test procedure to measure the static modulus of elasticity. While there will be some preliminary loading cycles to take out 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 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 EN 12390 test.

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

- using ultrasonic pulse velocity measurements. EN 12504-4 does not include a procedure for converting the ultrasonic pulse velocity readings into an initial tangent modulus, but the procedure is covered in BS1881-203. It is expected that this procedure will in included in the UK National Annex to EN 12504-4
- measuring the dynamic modulus by means of a variable frequency oscillator. A procedure for measuring the dynamic modulus (≈ initial tangent modulus) is given in BS1881-209.

6.3.2 Guidance on E-value testing

As deflection forms part of the serviceability limit state, an average E-value is appropriate and so the concrete mix proportions should be those that are expected to give the target mean compressive strength.

Care is needed when selecting a test machine to use for these 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 properly free to rotate. The existence of a problem may be identified if there are large differences between the three strain readings.

As a measured E-value is being used, the designer might 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. An even lower safety factor of unity is not recommended, as there will be uncertainty associated with the measured value and variability of production.

A number of research projects have reported that measured E-values are lower (sometimes significantly lower) than the value obtained by using the equation in the Eurocode. At the time of preparing this report, the reason for these differences is not known.

6.4 Enhancing the modulus of elasticity

There are a number of factors to be considered:

- generally, the E-value can be increased by increasing the characteristic strength – the principle behind the equation in **6.2.** However, to increase the modulus by 20% it is necessary to increase the concrete strength by at least three strength classes, a very expensive solution.
- since the bulk of the volume of concrete is aggregate, the E-value of the aggregate will have a significant effect on the E-value of the concrete. Selecting an aggregate with a high E-value will increase the modulus of elasticity of concrete. In most cases a producer will have no information on the aggregate modulus, but it is usually accepted that it is proportional to aggregate density.
- the E-value of concrete is a function of the E-values of its two constituents, paste and aggregate. Usually the E-value of the aggregate is the greater of the two, so increasing the aggregate volume can increase the E-value of the concrete - by around 5% [7]. When compared with the effect of aggregate type, this is a small increase, but mix design (relative volumes of aggregate and paste) is at least something over which the producer has control, whereas aggregate type often cannot be changed.

7. CREEP

7.1 Background

Creep is defined as the gradual increase in deformation (strain) with time for a constant applied stress, after taking into account other time dependent deformations not associated with the applied stress, i.e. shrinkage, swelling, thermal deformation, see Figure 5. For example, when the formwork from a slab is removed, the slab will deflect. This initial deflection is due to the elastic strain. However, with time the slab will progressively deflect more due to a number of factors, the main one being creep. As creep strain is typically 2.0 to 4.0 the elastic strain [8], the deflection due to creep can be several times larger than the initial deflection and therefore the designer must take this into account. Allowance for further movement needs to be made when fixing rigid, brittle partitions under concrete slabs and beams.

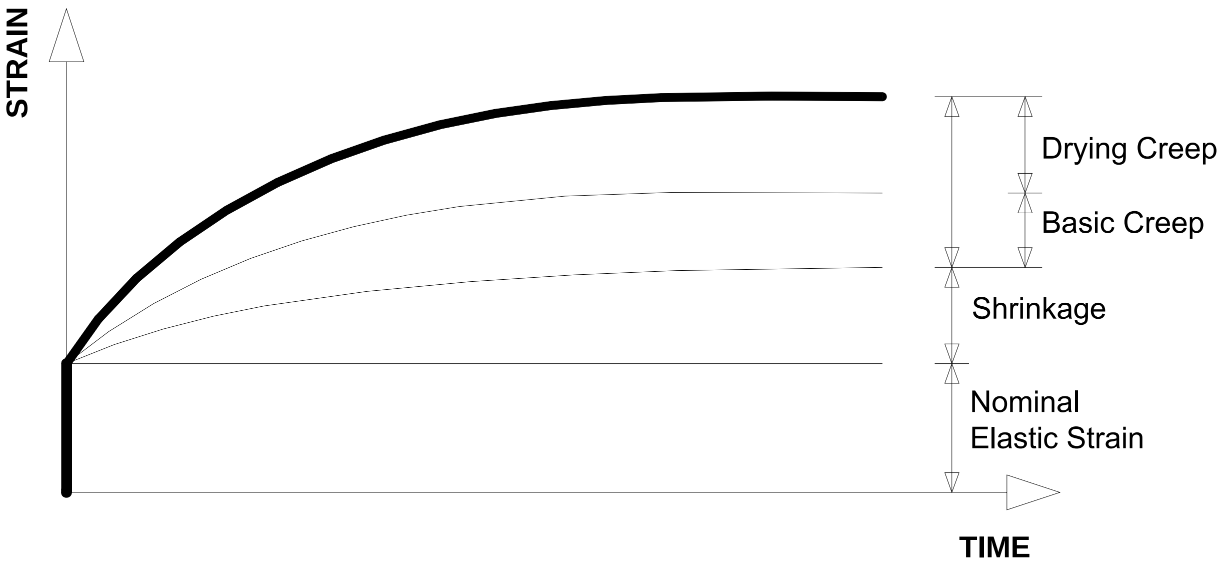


Figure 5: Time-dependent deformations in concrete subjected to a sustained load - change in strain of a loaded and drying specimen - *(after A.M.Neville, 'Properties of Concrete')*

Creep will also reduce the level of prestress in prestressed concrete elements, and in columns will cause a shortening of the column and a slow transfer of load from the concrete onto the reinforcement.

Creep in tension can also be beneficial in that it may partly relieve the stresses induced by other restrained movements, e.g. drying shrinkage, thermal contraction or by loading. While the mechanisms of tensile creep and compressive creep may be different, it is normal in design to assume the creep coefficients in tension and compression are the same [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.

7.2 How creep is dealt with in EN 1992-1-1

The **creep deformation** is calculated from a creep coefficient and the mean secant modulus of elasticity, and is given as:

$$\varepsilon_{cc}(\infty,t_0) = \varphi(\infty,t_0).(\sigma_c/1.05E_{cm})$$

where $\varepsilon_{cc}(\infty,t_0)$ = creep deformation at time t = ∞
$\varphi(\infty,t_0)$ = creep coefficient at time t = ∞
σ_c = constant compressive stress applied at time t = t_0

Note: the expression $1.05E_{cm}$ is actually the tangent modulus, E_c, which is taken simply as 1.05 times the secant modulus.

For indoor conditions (RH = 50%) and outdoor conditions (RH = 80%), figures are given for determining the **creep coefficient**, $\varphi(\infty,t_0)$, provided the concrete is not subjected to a stress greater than 45% of the characteristic strength at the time of loading. Where the applied stress exceeds this value there will be micro-cracking in the concrete that will increase the creep, and equations are provided in EN 1992-1-1 for taking this creep non-linearity into account.

7.3 Measurement of creep
7.3.1 Test methods

ASTM C 512-02 test is 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 get information relevant to a specific project. Six 150 mm 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, e.g. 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 2 to 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 [9], the equipment for the ASTM C 512 test is large and expensive and researchers tend to use smaller, less expensive equipment.

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

7.3.2 Guidance on creep testing

Some general considerations:

- concrete under stress will creep more if it is drying. The creep without drying is called 'basic creep' and the additional creep that is the result of the drying is called 'drying creep'. Therefore a creep test should be done in environmental conditions that reflect the in-use environmental condition.
- creep is also a function of the strength of the concrete at loading: the higher the strength at loading the lower the stress/strength ratio and the lower the creep.
- even with an equal stress/strength ratio, creep is lower when the load is imposed on the concrete at a later age. Therefore any test procedure needs to reflect the age at which the structure will be loaded.
- creep will increase with time, but after about 30 years it is assumed to be constant. Obviously it is not practical to test for such periods. About 50% of the ultimate creep occurs during the first 2 to 3 months and 90% after 2 to 3 years. The longer the testing period, the more accurate is the prediction of long-term creep. Gilbert [8] has reviewed the mathematical expressions for the shape of the creep coefficient versus time curves, and identifies the more useful expressions. He also concluded that the equations for predicting ultimate creep from 28-day creep test data were not reliable and a longer testing period is recommended.
- high temperatures have a significant effect on creep. However in the normal range of operating temperatures of structures the effect of temperature is relatively small compared to the effect of relative humidity.

For normal indoor conditions where project specific data are required, the standard requirements of ASTM C 512-02 apply, and a stress/strength ratio of 40% may be used. In some situations it will be necessary to vary the test in one or more of the following ways:

- a higher stress: strength ratio;
- a variable stress: strength ratio;
- a different temperature;
- a variable temperature;
- a different relative humidity;
- a variable relative humidity;
- the time of testing.

Testing conditions that reflect those likely to be encountered during construction should be selected and specified. The drying creep component of the total creep is a function of surface to volume ratio and as it is not normally practical to vary the specimen size, it may be necessary to take a worse case, or test at different relative humidity and interpolate for different parts of the section.

The constituent materials that are planned for the construction should be used in the test concrete, and the strength of the test mix should give a compressive strength in the range f_{ck} to (f_{ck} – 4).

Note 1: creep is higher in weaker concrete, even when the stress:strength ratio is the same and so using a strength lower than the mean value gives a small factor of safety.

Note 2: it may be necessary to increase the strength of the concrete if the mix design is controlled by other factors, e.g. maximum water/cement ratio.

Equation 1 of ASTM C 512-02 should be used to calculate the creep rate, and the creep deformation at, say, 30 years. The equation may also be used to calculate the creep coefficient (as opposed to assuming it, as in EN 1992-1-1) by dividing the creep deformation by the measured elastic strain.

7.4 Reduction of creep

Some considerations:

- a key factor in creep is the **stress/strength ratio at the time of loading**, so increasing the strength before loading can significantly reduce creep. Where the applied stress exceeds 45% of the characteristic strength at loading, non-linear creep may occur and this increases the creep.
- creep takes place in the cement paste and so an increase in the **volume of the aggregates** will reduce creep.
- concrete containing recycled aggregate will have an increased volume of cement paste and, as a result, an increased creep.
- creep is influenced by the **porosity of the cement paste** (w/c ratio) so high strength concrete will creep less than low strength concrete at an equal stress/strength ratio. This may, however, be offset by the effect of increased paste volume in high strength concrete. During hydration, the porosity of the cement paste is reduced and so for a given concrete and stress/strength ratio, creep reduces as the strength increases, i.e. as the time between casting and loading increases.
- as the aggregates restrain the creep in the cement paste, the stiffer the aggregate (higher E-value), the lower is the 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 why concretes containing fly ash or blastfurnace slag tend to have lower creeps.
- the presence of **reinforcement** can significantly reduce creep and this is taken into account during the design process. This aspect of reducing creep is not under the control of the concrete producer.
- on the assumption that the time of loading is unchanged, a producer does have some control over the value of the creep coefficient, as it is affected by cement type (or, more specifically, rate of strength gain), and the strength of the concrete.
- since creep deformation is a function of the E-value of the concrete, the factors affecting the modulus also affect creep.

8. SHRINKAGE

8.1 Background

Shrinkage is a combination of autogenous shrinkage and drying shrinkage.
Autogenous shrinkage occurs during setting and is caused by the internal consumption of water during hydration. The volume of the hydration products is less than the original volume of unhydrated cement and water and this reduction in volume causes tensile stresses and resulting shrinkage. In normal concrete, autogenous shrinkage is less than 100 microstrain, but in high strength concrete with a low w/c ratio, the autogenous shrinkage can exceed the drying shrinkage.
At a w/c ratio of 0.4 or higher, autogenous shrinkage is low enough to be ignored. At lower w/c ratios account should be taken of autogenous shrinkage. This is difficult in practice as there is no agreed test method or test period and the values reported in the literature vary widely from 200 to 1200 microstrain.
Drying shrinkage 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 are partially balanced by tensile creep relief.

The rate of drying shrinkage is dependent upon the relative humidity (RH) of the surrounding air and the 'notional size' (2x concrete cross sectional area / perimeter exposed to drying) of the section. As the RH of the air increases, the drying shrinkage reduces. As the notional size increases, the drying shrinkage reduces.

Taking the effects of shrinkage into account is a normal part of the design process and for this the shrinkage has to be estimated. If shrinkage is restrained, cracking is likely to occur. In prestressed concrete, shrinkage results in loss of prestress, and in asymmetrically reinforced concrete, it will increase deflection.

8.2 How shrinkage is dealt with in EN 1992-1-1

The shrinkage is taken as the sum of the autogenous shrinkage and the drying shrinkage.
8.2.1 The ultimate **autogenous shrinkage** is calculated from the specified characteristic cylinder strength and is given by the equation:

$$\varepsilon_{ca}(\infty) = 2{,}5(f_{ck} - 10) \times 10^{-6}$$

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

$$\varepsilon_{ca}(t) = \varepsilon_{ca}(\infty) \times (1 - \exp(-0{,}2t^{0{,}5}))$$

8.2.2 The nominal unrestrained **drying shrinkage** is calculated by a complex equation in EN 1992-1-1, Annex B or by looking up values in EN 1992-1-1, table 3.2. The data used to compile these equations is very old and the equation reflects old concrete technology where, for example, lower w/c ratios were achieved by using more cement and not by the use of admixtures. The Code states that the mean values given in the table have a coefficient of variation of 30%.
The term 'unrestrained' means unrestrained by reinforcement or by adjacent concrete

sections. The restraint given by the aggregate to the shrinkage of the cement paste is taken into account in the table. The table gives nominal unrestrained drying shrinkages ranging from 0,60% to 0,28% at a RH of 40%.

This nominal drying shrinkage depends upon the age at which a quantification is required, and on the notional size. The thicker the section, the lower the drying shrinkage at any given age.

The procedures in EN 1992-1-1 treat all concretes of the same grade in the same way, irrespective of aggregate type. There is no recognition of the high drying shrinkage that can occur when certain aggregate types are used.

8.3 Measurement of shrinkage

8.3.1 Test methods

Any **drying shrinkage test** on concrete will give the total drying shrinkage, i.e. the combined shrinkage of the cement paste and aggregate, when unrestrained by reinforcement.

The **ASTM C 157/C test method** for measuring drying shrinkage of concrete uses three 75 or 100mm prisms (depending on the maximum aggregate size) that are about 285mm long. After a day in the mould and a short period of conditioning in lime-saturated water, the length of the specimen is measured. It is then stored in lime-saturated water for a further 27 days and a second reading of length taken. They are then stored in a chamber at 23±2°C and 50±4 relative humidity for 64 weeks with length readings being taken at defined times. The rate of drying shrinkage will be a function of the specimen size.

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

Work has started on an ISO test (ISO/WD 1920-X), but this is at the committee draft stage and it is based on an Australian test procedure (AS 2350.13-1995).
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 equation 3.9 in EN 1992-1-1: 2003 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 humidity.

The **drying shrinkage of aggregates** is measured on concrete using the EN 1367-4 test. In the UK where aggregates with high drying shrinkage occur, the concrete standard places a drying shrinkage limit of 0.075% 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.

There is no European or international standard for the measurement of **autogenous shrinkage**.

8.4 Reduction of shrinkage

8.4.1 The potential **autogenous shrinkage** of normal concrete is low (<100 microstrain) and there is little benefit in trying to reduce it further. With high strength concrete made with a low water/cement ratio, the autogenous shrinkage may exceed the drying shrinkage. As the low w/c ratio and high cement content are determined by other requirements, there is little scope for reducing the autogenous shrinkage. However, if the paste volume can be reduced, the autogenous shrinkage will also be reduced.

8.4.2 **Drying shrinkage** is caused by the loss of water from the cement paste and in some cases from the aggregate. Aggregate content and properties are therefore important:

- the aggregate restrains the shrinkage of the cement paste, so the higher the volume of the aggregate the lower the shrinkage. Increasing the aggregate volume from 71 to 74% will reduce drying shrinkage by about 20% [9].
- the higher the E-value of the aggregate, the lower will be the drying shrinkage.
- if the aggregate is also shrinking, it will offer less restraint to the shrinkage of the cement paste and the drying shrinkage of the concrete will be higher. Where drying shrinkage needs to be minimised, aggregates with a low drying shrinkage should be used.
- an increase in the maximum aggregate size should result in a lower paste volume and thereby reduce the drying shrinkage, but this has to be balanced against possible increases in cover to reinforcement.
- using admixtures to achieve the required w/c ratio and consistence without increasing the cement content will reduce drying shrinkage [7].
- at a given aggregate volume, a concrete with a high w/c ratio will have a higher drying shrinkage than a concrete with a low w/c ratio [7].
- special admixtures can be used to reduce drying shrinkage.

9. THERMAL EXPANSION

9.1 Background

The **coefficient of thermal expansion** of concrete is a measure of the strain produced in concrete after a uniform change in temperature, where the concrete is neither restrained internally with reinforcing bars, nor has any external source of restraint. It is usually expressed in microstrain per degree centigrade and is typically in the range 8 to 13 microstrain / degree C.

As the temperature of concrete changes, it expands or contracts in response to that change. This has a number of structural implications, ranging from the need to provide joints to accommodate the movement of the structure, to the need for reinforcement to control crack widths when the thermal contraction is restrained. The temperature rise due to the release of the heat of hydration from the cement and additions (see section 11), and subsequent contraction on cooling, can lead to early-age thermal cracking [4]. If the resulting crack widths are to be controlled with reinforcement, the amount of reinforcement required is directly proportional to the coefficient of thermal expansion of the concrete. Reducing the coefficient of thermal expansion leads to a proportional reduction in the amount of crack control reinforcement required.

The coefficient of thermal expansion of concrete is not a constant as it varies with age and moisture content. Semi-dry concrete has a slightly higher coefficient of thermal expansion that saturated concrete [9].

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

The Code states that unless more accurate information is available, the coefficient of thermal expansion may be taken as 10 microstrain/ degree C.

9.3 Measurement of the coefficient of thermal expansion

There is **no standard method** for measuring the coefficient of thermal expansion in CEN, ISO or ASTM. There is a provisional standard written by the US transportation organisation, AASHTO, (TP 60-00 (2004), which is reported to require equipment not freely available.

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 at this temperature until this temperature is achieved throughout the specimen depth. A second set of readings is taken and the coefficient of thermal expansion calculated. The concrete mix proportions for the test should be those that are expected to give the target mean compressive strength, as this will err on the side of safety.

9.4 Reduction of the coefficient of thermal expansion

As the bulk of concrete comprises aggregate, using an aggregate with a lower coefficient of thermal expansion will reduce the coefficient of thermal expansion of the resulting concrete. Table 2 (from [4]) shows typical values.

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.

Table 2 Coefficients of thermal expansion of coarse aggregate and concrete

Coarse aggregate/rock group	Coefficient of Thermal expansion ($x\ 10^{-6}$/C)	
	Rock	Saturated concrete
Chert or flint	7.4 – 13.0	11.4 – 12.2
Quartzite	7.0 – 13.2	11.7 – 14.6
Sandstone	4.3 – 12.1	9.2 – 13.3
Marble	2.2 – 16.0	4.4 – 7.4
Siliceous limestone	3.6 – 9.7	8.1 – 11.0
Granite	1.8 – 11.9	8.1 – 10.3
Dolerite	4.5 – 8.5	average 9.2
Basalt	4.0 – 9.7	7.9 – 10.4
Limestone	1.8 – 11.7	4.3 – 10.3
Glacial gravel	-	9.0 – 13.7
Light weight (coarse and fine)	-	5.6 – 6.7
Light weight coarse and natural aggregate fines	-	7.0 – 9.5

10. FIRE RESISTANCE

10.1 Background

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.

The effects of fire on concrete are **loss of strength of the matrix** and **spalling** of the concrete skin. The loss of strength of concrete starts at around 300°C, but the main losses occur at temperatures of 500°C or more. Due to the slow heat transfer in concrete, the high temperatures are normally limited to the surface zone and so the section retains most of its strength. Spalling, can occur with most types of concrete, but the severity depends upon the aggregate type, concrete quality and moisture content. Sometimes explosive spalling can be caused by water turning into steam and increasing the vapour pressure in the pores within the concrete. As concrete quality increases, its ability to relieve the build-up of vapour pressure reduces and so spalling is more likely in high quality concrete. Even when spalling occurs, the integrity of the remaining concrete is usually adequate.

10.2 How fire resistance is dealt with in EN 1992-1-1

Information on fire resistance is given in EN 1992 Part 1-2, chapter 3 and also in EN1991-1-2: Fire actions. In Part 1-2 a distinction is made between siliceous and calcareous aggregates, the latter having the better performance at a given temperature

EN 1992-1-2 provides three methods of determining adequate fire resistance. These are: tabulated data, simplified calculation methods, and advanced calculation methods. For most buildings, the simple tables of proscribed axis distances will be used. The axis distance is the distance from the surface to the centre of the reinforcement. In special cases, fire engineering methods - where fire levels and resistance are calculated - may be used.

10.3 How fire resistance is measured

CEN has developed a suite of standard fire tests for the resistance, integrity and insulation of concrete elements. They cannot be used for concrete specimens.

10.4 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:

- 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 [10].
- calcium aluminate cement has a higher resistance to strength loss than other cement types. While this cement is widely used for non-structural applications, e.g. refractory linings, there is still debate over its suitability for structural applications and local provisions need to be followed.
- avoidance of high strength, low permeability concrete, which is more prone to spalling. However, other considerations are likely to over-ride that of fire performance and there is little practical scope for reducing concrete quality. Consideration should be given to including polypropylene fibres, see below.
- the use of polypropylene fibres has been shown to be effective in improving fire resistance. The mechanism is believed to be the fibres melting and being absorbed in the cement matrix; the fibre voids then provide expansion chambers for steam thus reducing the risk of spalling. There is still a need for further research to confirm the exact mechanism [10].

In fact, in design, the normal approach is to improve fire resistance of an element or structure rather than the concrete itself. The most widely-used approach is to increase the cover. This can be done directly by increasing the thickness of the concrete cover, or indirectly by using 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.

11. ADIABATIC TEMPERATURE RISE

11.1 Background

For the control of early-age thermal cracking, a specifier may specify a limit on the temperature rise of concrete. Compliance will generally require some initial testing of the proposed mix and/or some full-sized trials. As full-sized trials are expensive, the contractor may require the producer to undertake some initial testing of the concrete, and may specify a maximum adiabatic temperature rise. In practice adiabatic (i.e. completely insulated from ambient conditions) test facilities are not available and so the producer may use an approximation of the adiabatic temperature rise.

11.2 Measuring the approximate adiabatic temperature rise

The proposed concrete is cast into a low thermal mass mould, e.g. a thin plastic container that is insulated on all sides with at least 100mm thickness of expanded polystyrene (the thicker the better). The specimen size should be at least a 300mm cube. In the centre of this specimen is a thermocouple and the temperature over time is plotted. The temperature will rise over several days and eventually start to fall, as this is not a true adiabatic rig. The maximum temperature can be taken as being an engineering approximation to the adiabatic temperature rise.

11.3 Minimising the adiabatic temperature rise

As it is mainly the cement that produces heat, with some contribution from Type II additions, the keys to controlling the adiabatic temperature rise are to minimise the cement content and use cements with large proportions of secondary main constituents. Low heat and very low heat cements should be considered. If compressive strength controls the mix design, producers are advised to minimise the Portland cement clinker content, and use admixtures to reduce the w/c ratio to achieve the required strength. If achieving the required consistence is a problem, the use of filler aggregate might be preferable to increasing the cement content.

12. PRACTICAL ASPECTS OF SUPPLY

If asked to supply concrete with special requirements for these engineering properties, concrete producers may be coming across the requirements and specialised test methods for the first time. Equally, designers / specifiers and contractors may also have little experience in their use, or in the demands that are likely to be placed on the supplier, so the producer – purchaser relationship is a particularly important one in these circumstances. The producer should not underestimate the amount of work and time involved in attempting compliance with these specialised requirements, and this should be made clear to the purchaser.

Production. The purchaser may not understand the limitations of ready-mixed concrete production. He may not understand, for example, that a producer may have operational difficulties in producing concrete with an aggregate which is not normally held in stock at a plant, especially for small quantities. He may not appreciate the particular difficulties (and risk, and cost) involved in supplying small quantities of highly specified concrete, even though the concrete might not be a completely new mix.

Testing. It is clear from this document that a lot of the testing which may be necessary is non-standard. Requirements for testing (and for interpretation of results) should be clearly specified and agreed in advance, and the limitations on any selected method made clear to the purchaser, who may not be familiar with test variability, or with concepts like (for example) mean and characteristic values for properties.

In general, a producer might regard this as an opportunity to get across to customers a better understanding of the technical competence of the industry, but also of the realities of concrete production.

13. IDENTIFIED NEED FOR FURTHER RESEARCH AND DEVELOPMENT

In a number of areas there is no defined procedure and test method for determining basic design values experimentally. Further research and development is needed to provide a test and procedure to measure:

- creep coefficient;
- ultimate autogenous shrinkage;
- nominal drying shrinkage of concrete;
- coefficient of thermal expansion.

From a commercial viewpoint consideration should be given to an examination of potential benefits of the use of providing real design values based on real materials proposed for any project, rather than the 'safe' European values.

14. STANDARD METHODS FOR MEASUREMENT OF PHYSICAL PROPERTIES

PROPERTY	STANDARD TESTS	COMMENTS
Compressive strength, cylinders and cubes	EN 12390-3	
Tensile splitting strength	EN 12390-6	
Flexural strength	EN 12390-5	
Axial tensile strength	No standards	
Static modulus of electricity (secant modulus)	Standard EN test under development	
Dynamic modulus of elasticity (≅ initial tangent modulus)	EN 12504-4 BS1881-209.	This EN, for determination of ultrasonic pulse velocity, upv, does not yet describe conversion of upv into an initial tangent modulus. Procedure may be given in national annexes 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
Drying shrinkage of concrete	No standard EN test. ASTM C 157/C ISO/WD 1920-X	
Drying shrinkage of aggregate	EN1367-4	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	

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European and other national standards

EN 1992-1 Eurocode 2:Design of concrete structures
Part 1-2 – General rules and rules for buildings
Part 1-2 – General rules – Structural file design.
EN 197-1 Composition, specifications and conformity criteria for common cements.
EN 206-1 Concrete – Specification, performance, production and conformity.
EN 12390-3 Compressive strength of test specimens.
EN 12390-5 Flexural strength of test specimens.
EN 12390-6 Tensile splitting strength of test specimens.
EN 12504-4 Determination of ultrasonic pulse velocity.
EN 12617-4 Products and systems for the protection and repair of concrete structures – Test methods – Part 4: Determination of shrinkage and expansion.
prEN 13584-2 Products and systems for the protection and repair of concrete structures – Test methods – Part 2: Determination of creep.
ENV 13670 Execution of concrete structures.
EN 13791 Assessment of concrete compressive strength in structures or in precast concrete products.

BS EN 1367-4 Determination of drying shrinkage. (Note: for aggregates.)
BS 1881-203 Recommendations for measurement of velocity of ultrasonic pulses in concrete.
BS 1881-209 Recommendations for the measurement of dynamic modulus of elasticity

ASTM C 157/C Standard test method for length change of hardened hydraulic-cement mortar and concrete.
ASTM C 512-02 Standard test method for creep of concrete in compression.
US transportation organisation, AASHTO, TP 60-00 (2004) Coefficient of Thermal Expansion of Hydraulic Cement Concrete.

AS 2350.13-1995 : Methods of testing Portland and blended cements - Determination of drying shrinkage of Portland and blended cement mortars.

International standards

ISO/WD 1920-X Testing concrete – Part X: Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
ISO/WD 1920-Y Testing concrete – Part Y: Determination of creep of concrete cylinders in compression

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