BS EN 15080-8:2009

BSI Standards Publication

Extended application of results from fire resistance tests

Part 8: Beams

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National foreword

This British Standard is the UK implementation of EN 15080-8:2009.

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A list of organizations represented on this committee can be obtained on request to its secretary.

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Management Centre: Avenue Marnix 17, B-1000 Brussels

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Foreword

This document (EN 15080-8:2009) has been prepared by Technical Committee CEN/TC 127 “Fire safety in buildings”, the secretariat of which is held by BSI.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by April 2010, and conflicting national standards shall be withdrawn at the latest by April 2010.

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1 Scope

This part of EN 15080 identifies the parameters and factors that affect the fire resistance of beams and need to be taken into account when considering extended application of results of beams tested in accordance with EN 1365-3. It also gives the methodology to be used when preparing an extended application, including rules and calculation methods which can be applied to establish the resultant influence of a variation in one or more parameters and to determine the field of extended application.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 338, *Structural timber — Strength classes*

EN 1194, *Timber structures — Glued laminated timber — Strength classes and determination of characteristic values*

EN 1363-1:1999, *Fire resistance tests — Part 1: General Requirements*

EN 1365-3:1999, *Fire resistance tests for loadbearing elements — Part 3: Beams*

EN 10025-1, *Hot rolled products of structural steels — Part 1: General technical delivery conditions*

EN 10025-2, *Hot rolled products of structural steels — Part 2: Technical delivery conditions for non-alloy structural steels*

EN 10080–1, *Steel for the reinforcement of concrete — Weldable reinforcing steel — Part 1: General requirements*

prEN 10138-1, *Prestressing steels — Part 1: General requirements*

EN 13501-2*, Fire classification of construction products and building elements — Part 2: Classification using data from fire resistance tests, excluding ventilation services*

EN ISO 13943:2000, *Fire safety — Vocabulary (ISO 13943:2000)*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in EN ISO 13943:2000, EN 1363-1:1999 and EN 1365-3:1999, together with the following apply.

3.1
test result
outcome of a testing process and its associated procedures detailed within EN 1365-3 (which may include some processing of the results from the testing of a number of specimens). A test result is expressed in terms of one or more fire performance parameter(s)

3.2
direct field of application of test results
outcome of a process (involving the application of defined rules) whereby a test result is deemed to be equally valid for variations in one or more of the product properties and/or intended end use application(s)

NOTE The direct field of application of test results are presented in EN 1365-3.

3.3
extended field of application of test results
outcome of a process (involving the application of defined rules that may incorporate calculation procedures) that predicts, for a variation of a product property and/or its intended end use application(s), a test result on the basis of one or more test results to the same test standard, i.e. to EN 1365-3

3.4
classification
process defined in EN 13501-2, whereby the fire performance parameters obtained from the results of one test, or a set of tests, or from a process of extended application, are compared with limiting values for those parameters that are set as criteria for achieving a certain classification

NOTE The relevant classes and related criteria for fire resistance are specified in Commission Decisions (2000/367/EC, 2000/147/EC and 2001/671/EC).

3.5
reference test
fire resistance test according to EN 1365-3 on a beam from which the test result is used for the process of extended application

NOTE There may be more than one reference test.

3.6
parameter
aspect of the reference scenario that may vary in practice and may result in a change of the fire resistance performance

NOTE Examples are the load level and the span.

3.7
modelling factor
factor determined for a considered relevant structural failure mode on basis of the assessment of the reference test(s), which takes into account the differences between the test results and calculated results, and which is used to adjust the results of the extended application

3.8
calculated structural resistance
resistance to bending or shear of a beam in a fire test calculated at the end of the test

3.9
effective structural resistance
predicted resistance to bending or shear of a beam for use in an extended application

3.10
relative resistance
ratio of the bending or shear resistance of a beam in a fire resistance test to the resistance at normal temperatures calculated with all safety factors taken as unity

3.11
target classification
fire resistance that the extended application is required to achieve

4 Basis and methodology of establishing the extended application

4.1 General

An extended application analysis is required when the application of a beam is not covered by the field of direct application given in the classification document of the product.

The situation of (a) fire test(s) carried out according to EN 1365-3 will be referred to as the “reference test” and “reference scenario”. The result of a test, i.e. the fire resistance with respect to the load bearing capacity, will be referred as “$t_{ref,fi}$”.

If more than one reference test is available, all the tests may be used for the extended application provided that the tests all have the same mechanical boundary conditions and have all been carried out using the same fire curve.

NOTE It is possible that in the classification report all reference beams are classified with the same classification “R_{ref}” although the actual test results ($t_{ref,fi}$) given in the test reports may differ.

4.2 Basic principles

4.2.1 General

It is assumed that extended application is made by appropriately qualified and experienced persons in the field of structural fire design.

The reference test(s) shall be well documented, i.e. an insight into the performance of the test specimen(s) and the mode of failure, leading to R_{ref}, are available.

Three analyses (described in 4.3, 4.4 and 4.5) should be carried out where appropriate. It shall be decided whether:

— field of application can be extended, maintaining the classification R_{ref} or changing the classification and if so, by how much;

— extension is not possible (new tests are required).

Any predicted increase in fire resistance shall not exceed the lesser of 15 min and 20 % of the target classification.

NOTE This is illustrated in A.3.

4.2.2 Basis of the extended application

An adequate understanding of the structural and thermal performance, as well as an understanding of other relevant features, shall be achieved based on the scope of the required extended application. For minor or obvious extensions to the reference test, the depth of analysis required may be reduced.

4.2.3 Mode of failure

Any assessment shall consider the possibility that the mode or cause of failure, such as structural collapse in bending or failure of a fire protection system, might change and that the mode or cause of failure in a fire test may no longer be critical if one or more parameters are changed.

If a change of failure mode is expected, then extended application is not possible unless additional information is available.

NOTE For additional information see A.1.

4.2.4 Methods of analysis

When analysing the reference test(s), the rules given in the Eurocodes shall be used if applicable. Additional rules are given in this standard. These are also applicable in cases where the Eurocodes do not fully cover the construction to be assessed. Other calculation models, as well as empirical rules, shall be validated on the basis of similar tests as the reference test(s). Historic data and ad hoc tests may be used to supplement to the information of the reference test(s).

4.3 Basic thermal analysis

If the extended application is intended to be for a cross section of a size or shape different from the reference test(s) or for a different resistance time or another nominal fire curve, then a thermal assessment shall be made. The analysis should lead to an understanding of the temperature distribution and material strength variation throughout the beam.

The analysis may take the form of a finite element or finite difference thermal analysis. In limited circumstances, when a dimension is changed, it may be possible to show, using a simple calculation, that the temperature distribution measured in the test can be conservatively used for the modified cross section.

For timber beams, it may be sufficient to analyse the charring depth instead of carrying out a complete thermal analysis. Where a thermal analysis is carried out, the position of the char-line shall be taken as the position of the 300 °C isotherm.

4.4 Basic structural analysis

4.4.1 General

The structural behaviour of the reference test(s) and of the situation to be assessed shall be analysed. The depth of structural analysis will depend on the complexity of the beam and the extent of the proposed extended application. For any assessment the same failure modes states should be considered as were considered for normal temperature design. These include:

— Bending (including lateral torsional buckling).

— Vertical shear.

— Horizontal shear.

NOTE 1 It is not normally necessary to consider deformations. See A.7.

The assessment shall also include:

— Connections, either mechanical or glued, between parts of the construction.

— Boundary conditions.

— Material properties versus temperature.

NOTE 2 As an illustration of the depth of structural analysis required, the report on the analysis for the vertical shear check on a steel beam might simply say, “The shear is low enough to have no influence on the bending strength - no check required”.

4.4.2 Modelling factor

Any assessment shall take into account the accuracy of the structural model used. Models which overestimate the load resistance of the reference test(s), shall have a modelling factor applied when used to make an assessment for extended application.

In making any assessment, the effective structural resistance shall be determined as follows:

$$R_{\mathrm{eff}} = R \times k_{\mathrm{mf}} \tag{1}$$

where

R_{eff} is the effective structural resistance;

R is the calculated structural resistance;

k_{mf} is the modelling factor.

For a single reference test, the modelling factor is defined as:

$$k_{mf} = \frac{F}{R} \quad \text{but not greater than 1,0} \tag{2}$$

where

F is the applied load or moment in the reference test.

For more than one reference test:

$$k_{mf} = \frac{1}{n}\sum\frac{F_i}{R_i} \quad \text{but not greater than 1,0} \tag{3}$$

where

F_i is the applied load or moment in the reference test *I*;

R_i is the calculated structural resistance of test *i* calculated using the measured temperature distribution;

n is the number of tests.

and with each individual value:

$(F_i / R_i) \leq 1,3$

If the value of (F_i / R_i) exceeds 1,3 then that test should not be used as part of the assessment of extended application.

NOTE 1 If the measured temperature distribution is not sufficiently comprehensive to allow the structural resistance to be adequately predicted then it may be supplemented with computed temperatures.

NOTE 2 If the value of (F_i / R_i) exceeds 1,3, there may be something wrong with the test data or with some aspect of the engineering model being used so the particular test may be considered unreliable.

4.4.3 Material properties

The reference test(s) should be assessed using measured material strength. If the actual material strength is not available then the strength should be taken from Table 1.

In making any assessment of extended application, mean material properties shall be assumed. If the actual mean value is unknown, then values for the mean strength should be taken from Table 1.

NOTE The values in Table 1 are conservative values for the mean strength.

For additional information see A.1.

Table 1 — Conservative values for mean strength

Material	Mean strength[(a)]
Concrete compressive strength	f_{ck} + 8 N/mm[2]
Reinforcement steel to EN 10080 for concrete	$f_{sk} \times 1{,}1$
Prestressing steel to EN 10138-1 for concrete	$f_{pk} \times 1{,}1$
Hot rolled structural steel to EN 10025 parts 1 and 2	$f_y \times 1{,}1$
Timber - Structural timber, strength classes C14 to C40 to EN 338 - glued laminated timber, all strength classes to EN 1194	 $f_k \times 1{,}5$ $f_k \times 1{,}3$
- LVL	$f_k \times 1{,}25$
Other materials	to be estimated
(a) $f_{i\,k}$ and f_y are the characteristic strength values (5 % fractile).	

4.5 Analysis of other features

Where relevant phenomena not assessed in 4.3 and 4.4 shall be taken into account.

NOTE These may include such things as the stickability of fire protection materials and spalling of concrete. For spalling of concrete, reference should be made to EN 1992-1-2:2004, subclauses 4.5 and 6.2.

The stickability of fire protection materials should be either:

assessed using the methods given in ENV 13381,

or

be based on clear evidence of the performance of the material and fixing system in at least two fire tests.

5 Critical parameters

5.1 General

The parameters listed in 5.2 to 5.6 may affect the fire performance, i.e. the value of $R_{ref,}$ and shall be taken into account when preparing an extended application. The specific constructional parameters vary depending upon the nature of the beam being considered.

NOTE 1 The list in this section is not definitive; in special cases other parameters may also be appropriate.

NOTE 2 The review in Annex A of parameters and of corresponding factor and factor influences, is only given for the common thermal, common mechanical and common constructional parameters.

5.2 Common thermal parameters

1) Nominal gas temperature time curves.

2) Number of exposed faces.

5.3 Common mechanical parameters

1) Mechanical load.

2) Distribution of mechanical load.

3) Axial restraint.

4) Rotational restraint.

5) Lateral restraint.

5.4 Common constructional parameters

1) Span.

2) Dimensions of cross section.

3) Shape of cross section.

4) Surface dimensions of bearings.

5) Position and size of holes.

5.5 Specific constructional parameters for beams without applied fire protection

5.5.1 Concrete beams

The following specific parameters can be distinguished:

a) Concrete specification

NOTE 1 This will normally include:

1) Type of concrete, i.e. normal weight or lightweight concrete.

2) Type of concrete aggregate, i.e. siliceous, calcareous or lightweight.

3) Strength of concrete.

b) Specification of reinforcement

NOTE 2 This will normally include:

1) Type of reinforcement, i.e. reinforcing steel, prestressing bars, wires or strands.

2) Characteristic strength of the reinforcing or prestressing steel.

3) Is the steel hot rolled or cold worked or quenched and tempered?

4) The ductility characteristics of the reinforcing steel.

c) Degree of any prestressing.

d) Bond and anchorage properties of the reinforcing or prestressing steel, i.e. ribbed or plain surface.

e) Bonded versus unbonded tendons (post tensioned beams).

f) Amount of the main reinforcement and position of the main reinforcement in the cross section.

g) Concrete cover to reinforcement.

h) Amount and position of the shear reinforcement.

i) Minimum dimensions, the level of compressive stresses, moisture content and other parameters possible relevant for spalling.

j) Other parameters which may effect spalling.

5.5.2 Steel beams

The following specific parameters can be distinguished:

1) Steel specification.

2) Section factor.

3) Classification of cross section (see EN 1993-1-2).

4) Web stiffeners.

5.5.3 Composite steel-concrete beams

In addition to the items listed in 5.5.1 and 5.5.2, the following parameters may apply:

1) Degree of shear connection.

2) Number of shear connectors.

3) Type of shear connectors.

4) Type of concrete slab.

NOTE This could include a consideration of whether the slab is flat or composite with profiled steel sheeting.

5) Thickness of the concrete slab.

5.5.4 Timber beams

The following parameters apply, where relevant, to timber beams made of solid timber, glued laminated timber and LVL (laminated veneer lumber) or other engineered wood products:

a) Timber specification

NOTE This will normally include:

1) Material (timber or wood based materials).

2) Species of timber.

3) Strength class of solid or glued laminated timber (including characteristic values of strength and stiffness parameters and density).

4) Strength.

5) Stiffness.

6) Density.

7) Charing rat.

b) Type of adhesive

The above parameters may also apply to beams composed of several parts connected by gluing, such as thin-webbed I-beams with solid timber flanges, thin-flanged beams with solid timber webs and I-beams with solid or glued laminated timber flanges and webs. For these beams, in addition to the above, the following parameter may also be applicable:

c) Materials of flange(s) and web(s).

5.5.5 Mechanically jointed beams

The parameters presented in 5.5.4.1 and 5.5.4.2 apply where relevant also to beams of box-, I- or T-shape, composed of several parts connected by mechanical fasteners with flanges and/or webs made of solid or glued laminated timber, wood-based panels and LVL or other engineered wood products. In addition, the following parameters may apply:

1) Type of fastener.

2) Spacing of fasteners.

3) Edge distances of fasteners.

5.6 Specific constructional parameters for beams with applied fire protection

In addition to the parameters given in 5.5, the following parameters may apply:

1) Surface treatment of the beam (i.e. grit blasted, primer).

2) Type of protection material (generic or proprietary).

3) Thermal characteristics of the protection (thermal conductivity and specific heat).

4) Density of the protection.

5) Thickness of the protection.

6) Number of layers of the applied fire protection.

7) Position and geometry of joints in the applied fire protection.

8) All details of method of fixing of the fire protection (type of fasteners, spacing and edge distances; including reinforcement where relevant).

9) Surface treatment of the fire protection.

10) Position of the fire protection (vertical, horizontal sides; one or more sides may be unprotected).

6 Report of the extended application analysis

The Extended Application report shall be used in conjunction with the classification document as specified in EN 13501-2 and shall contain the following:

1) Name of the sponsor.

2) Name of the expert body which has performed the extended application.

3) Date of issue of the report.

4) Unique reference number for the report.

5) Type of the tested beams. This shall include a general description and any trade names of all the products involved.

6) Scope of the extended application.

7) Summary of the report(s) of the reference test(s) and any previously granted extended applications, if available.

NOTE This is a specially prepared summary and not necessary the brief summary sometimes given as part of the test report or as a separate document. Alternatively, it is acceptable to append full copies of the relevant reports of the reference test(s).

8) Analysis of the extended application, writing including:

— Source of any calculation model used.

— Justification of the use of any calculation model for this particular extended application.

— List of any assumptions made together with a justification for those assumptions.

— Any supporting information and test references from other fire resistance tests (to European Standards) or from historic fire resistance tests (to National Standards) or from ad hoc or small-scale tests.

9) Conclusion of the analysis, including the new classification of the fire resistance (R).

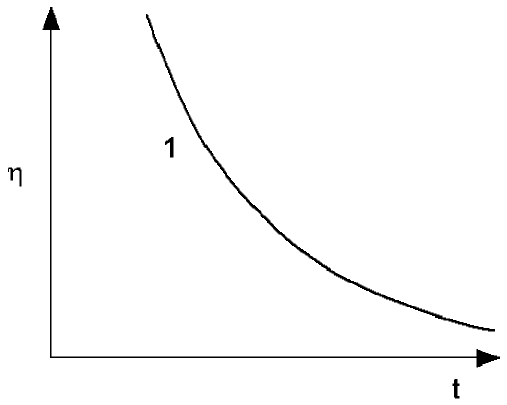
Annex A
(informative)

Guidelines for making assessments

A.1 Mode of failure

A.1.1 General

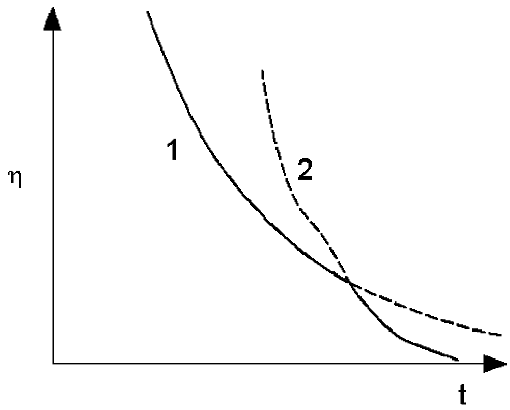
Care should be taken to ensure that any change in the mode of failure should be taken into account when assessing an extended application. For a simple beam the effect is shown diagrammatically in Figure A.1 and Figure A.2. The two modes might be bending and shear.



Key

η	Relative resistance
t	Time
1	Failure mode 1
2	Failure mode 2

Figure A.1 — Member resistance as a function of time for a single mode of failure



Key

η	Relative resistance
t	Time
1	Failure mode 1
2	Failure mode 2

Figure A.2 — Member resistance as a function of time for a changing mode of failure

Other examples of a change in the mode of failure.

A.1.2 Failure of protection system

A beam achieves t = 52 min. If the applied loading is reduced R60 may be possible. However, large cracks were observed in the protection system at the end of the test which may indicate a potential stickability failure of the protection system. It is important to take into account the stickability of any protection system as it will affect the heating rate of the beam. The stickability may be affected by any increase in thickness of fire protection.

A.1.3 Change of structural mode of failure from bending to shear

A fabricated steel beam containing a large service opening is tested. A load-bearing failure occurs caused by bending at mid-span. Calculations indicate that the web of the beam could be reduced in thickness as this will only have a small effect on the bending strength. However, a thinner web will have less shear resistance which may cause shear failure at the opening and may also heat up more quickly.

A.1.4 Change of structure mode of failure from bending to connection failure

A beam is tested and achieves t = 79 min. Calculations indicate that R90 could be achieved if the applied loading was reduced. However, the connection between two parts was highly stressed and was operating at a higher utilisation than the parts in bending. The possibility that a failure could occur before 90 min, even with reduced loading, because of failure of the connection should be considered.

A.2 Effect of material strength

Structural designs are normally based on characteristic material strengths. Fire tests are carried out on test constructions in which the material strength of each component is likely to be higher than the characteristic value. Generally, higher material strengths will lead to higher fire resistances in terms of load bearing capacity.

In making an assessment of the test, the reason why a particular performance was achieved might be wrongly identified. For example, it might be assumed that a connection was carrying a high proportion of the applied load when, in reality, one of the other components was actually supporting more of the load.

For example, consider a fire test on a reinforced concrete beam which achieves t = 82 min. The characteristic and assumed actual material properties are:

Material	Characteristic strength (N/mm^2)	Actual strength (N/mm^2)
Concrete	30	38
Steel reinforcement	500	585

As the strength of reinforcement is normally the dominant parameter in calculating the strength of the beam, a false impression of how well the beam performed in the test may be obtained. The actual bending strength could have been 117 % (585/500) of the strength based on characteristic reinforcement strength. Although the mean rate at which strength was lost was about 1,2 % per min, beams do not lose strength linearly with time in a fire resistance test so, during the last 15 min, the rate was probably about 3 % per min. The extra 17 % of strength might therefore represent about 6 min. A beam with only characteristic material properties might therefore only achieve t = 76 min (= 82 - 6).

In view of the effect of material properties on fire resistance, it is recommended in 4.4.3 that tests are assessed using measured values for all material properties. It follows that if it is known a test is going to be used as a reference test for extended application then the material properties should be measured. Alternatively, if material properties are not known, it is recommended that mean values should be assumed. Subsequent predictions for an extended application should be made assuming mean material properties. For the common structural materials, these are presented in Table 1.

For structural steel, reinforcement and concrete is it not possible to give definitive values for mean material strength as the product standards do not specify mean strength. For these materials, the mean strength values given in Table 1 are conservative estimates. For timber, the product standards do specify mean strength values and these are reproduced in Table 1. For materials not included in the table a conservative estimate of mean strength should be used.

A.3 Extrapolation of fire resistance

In 4.2.1 it is stated that any increase in predicted fire resistance is limited to the minimum of 15 min or 20 % of the target classification fire resistance in any of the reference tests. The effect of this is illustrated in Table A.1.

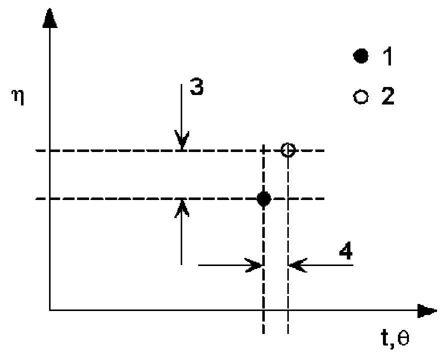
It should be noted that 4.2.1 specifies a *maximum* extension to fire resistance that an extended application can be applied to. In many cases the consideration of the mode or cause of failure (see A.1) will govern. The mode or cause of failure will be particularly important when considering, for example, the charring of timber or the stickability of a fire protection material.

Table A.1 — Maximum increase in fire resistance

Target Classification (min)	Minimum fire resistance required in the longest duration reference test (min)
30	24
45	36
60	48
90	75
120	105
150	135
180	165
240	225

A.4 Accuracy of predictions

The way of expressing the accuracy of any prediction will affect its numeric value. An error of 10 % in fire resistance time will probably not correspond to a 10 % error in beam strength. For example, steel looses strength very quickly with increasing temperature so a 5 % error on steel temperature at 600 °C (30 °C) might correspond to a 20 % difference in steel strength. However, if the steel reaches 600 °C in 60 min, towards the end of the test, it might be heating at about 14 °C per min. The difference in fire resistance will be about 2 min. Care should therefore be taken when considering differences between the test result and any prediction. Greatest importance should be placed on any variations in fire resistance (see Figure A.3).



Key

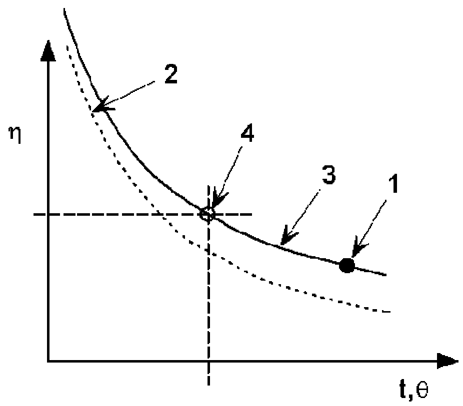
η Relative resistance
t, θ Time or temperature
1 Test result
2 Prediction
3 Error in terms of relative resistance
4 Error in terms of time or temperature

Figure A.3 — Illustration of types of error in predictions

A.5 Prediction based on material laws

The relationship between load carrying ability and temperature (time) will often be similar to the material law governing the key or critical element. This is illustrated in Figure A.4.

The curve describing the material law is drawn in terms of the strength retention factor against temperature. The test point is then plotted with the applied load converted to a fraction of normal strength and the temperature at that of the critical element. This point will normally be above the curve for the material law. The material law curve is then moved vertically until it passes through the test point. This modified curve may be used to estimate the load carrying capacity of the beam relative to the temperature of the critical element, assuming that the mode of failure does not change (see A.1).

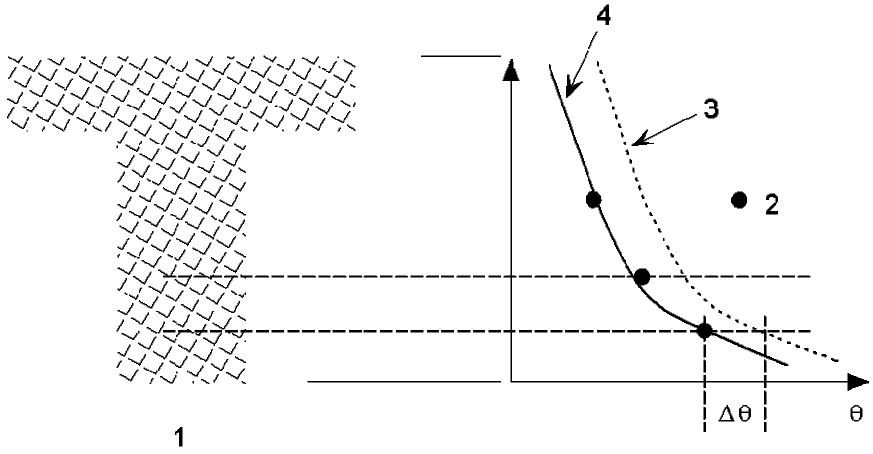


Key
η Relative resistance
t, θ Time or temperature
1 Test result
2 Material law
3 Curve drawn by moving the Material Law vertically
4 Prediction

Figure A.4 — Illustration of prediction based on material model

A.6 Modifying predicted temperatures

Computer modelling of the temperature distribution may not always accurately predict the temperature of certain key elements of the beam but may often predict the relative values of temperature reasonably well. A useful technique to obtain a temperature distribution that can be used in an assessment is to logically adjust the predicted temperatures so that they align with the key measured temperatures. The adjusted predicted temperature distribution is then be used. This is illustrated in Figure A.5. Any adjustment should take account of the effect of errors on the structural resistance. The temperature of a reinforcement bar towards the bottom of a beam will be structurally much more important than the temperature of a concrete element at the top of a beam.



Key

θ	Temperature
1	Beam cross section
2	Measured temperatures
3	Predicted temperatures
4	Adjusted prediction
Δθ	Temperature adjustment

Figure A.5 — Illustration of adjusted temperature distribution

A.7 Deflection limits

Deformation criteria do not normally need to be considered separately in the EXAP assessment. Where fire protection of the beam contributes to the fire resistance of the beam, it shall be verified that the fire protection remains cohesive and coherent when subjected to the strain and temperature, which result from any change in the parameters.

NOTE The main reason to introduce deformation limits into EN 1365-3 is to protect people carrying out fire tests from the effect of a sudden beam collapse into the furnace and to limit potential damage to the furnace, rather than avoiding problems in the end use situation. In the fire parts of the Eurocodes a similar approach is adopted. For example, Eurocode 3: Design of steel structures, Part 1.2: General rules, Structural fire design, states:

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1) Where mechanical resistance in the case of fire is required, steel structures shall be designed and constructed in such a way that they maintain their load bearing function during the relevant fire exposure.

(2) Deformation criteria shall be applied where the protection aims, or the design criteria for separating elements, require consideration of the deformation of the load bearing structure.

(3) Except from (2) consideration of the deformation of the load bearing structure is not necessary in the following cases, as relevant:

- *the efficiency of the means of protection has been evaluated according to section 3.4.3;*

and

- *the separating elements have to fulfil requirements according to a nominal fire exposure.*

Annex B
(informative)

The Extended Application Of Steel Beams

B.1 Introduction

This annex illustrates the use of extended applications of the results of two reference tests on fire protected steel beams.

The reference tests have been performed in accordance to EN 1365-3. Bending failure occurred. A lightweight insulation material was applied. Evidence is available that the fire protection system complies with ENV 13381-4. No steel temperatures are reported. Apart from the load level, the two reference tests are identical.

During both reference tests, the mechanical loading was kept constant till failure occurred. The degree of utilisation μ_0, defined as the ratio between the applied load and the failure load under room temperature conditions (calculated on basis of the actual yield stress of the steel), amounts:

reference test 1: μ_0 = 0,60;

reference test 2: μ_0 = 0,70.

The reference tests resulted in the following times of fire resistance with respect to the load bearing capacity:

reference test 1: $t_{\text{ref, fi}}$ = 86 min;

reference test 2; $t_{\text{ref, fi}}$ = 75 min.

During the tests, the insulation material remained coherent and adhesive during the full duration of the tests. Directly after reaching the performance criteria, the tests were terminated.

The client needs to meet a fire resistance requirement of 90 min and wishes to know the load reduction, necessary to meet this requirement. To this end, extended application is carried out on the basis of the results of the two reference tests, taking into account semi-empirical rules from literature for the heating behaviour of insulated steel members.

B.2 Analysis of reference tests

B.2.1 Thermal performance

Analogous to the assumptions in EN 1993-1-2, a uniform temperature distribution across and along the steel beams is assumed.

The time t, to reach a temperature of θ_a ºC in the steel profile is assessed by the following semi-empirical Equation (B.1). The equation is fully described in *Design Manual on the European Recommendations for the Fire Safety of Steel Structures,* (published by ECCS, see bibliography):

$$t = 40 \cdot (\theta_a - 140) \cdot \left(\frac{d}{\lambda} \cdot \frac{V}{A} \right)^{0,77} \qquad \text{(B.1)}$$

where

A is exposed surface of the steel member;

V is volume of the steel member;

d is thickness of the applied insulation material;

λ is thermal conductivity of the applied insulation material.

The factor [d/λ*V/A] reflects the combined effect of the section factor (A/V) and the insulation parameter (d/λ) on the heating rate and is called the Heating Rate Parameter (HRP).

The application range for the model on which Equation (B.1) is based, is specified as follows:

30	<	t	<	240	min	(B.2a)
33×10^{-5}	<	HRP	<	1000×10^{-5}	m^2C/W	(B.2b)
400	<	θ_a	<	600	ºC	(B.2c)

B.2.2 Mechanical performance

The mechanical performance is based on the EN 1993-1-2, in which the relation between the degree of utilisation and the critical steel temperature of a steel beam is given by:

$$\theta_{\mathrm{a,cr}} = 39{,}19 l_{\mathrm{n}}\left[\frac{1}{0{,}9647\mu_0^{-3{,}833}} - 1\right] + 482 \tag{B.3}$$

where

$\theta_{a,cr}$ is the critical steel temperature of the beam;

μ_0 I s the degree of utilisation of the steel beam.

Substituting Equation (B.1) in Equation (B.3) renders the time to reach the critical steel temperature under standard fire conditions as function of the degree of utilisation. This time can be identified as the fire resistance with respect to the load bearing capacity of a steel beam (= t_{fi} (μ_0)). Hence:

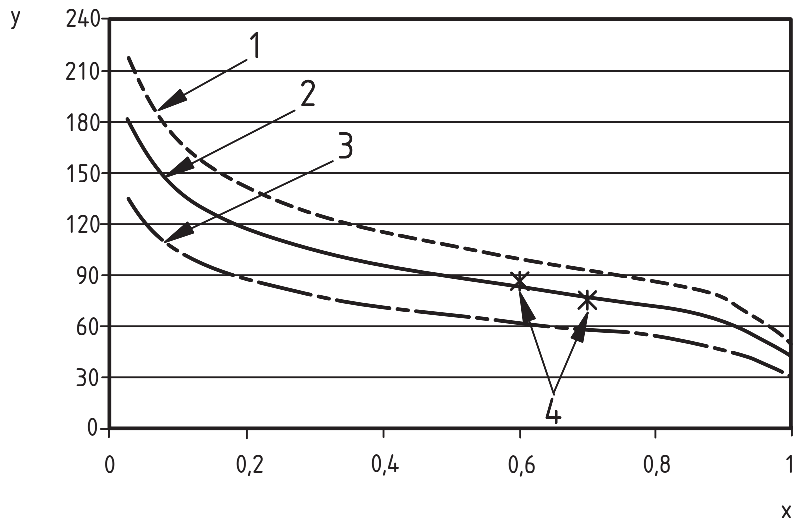
$$t_{\mathrm{fi}}(\mu_0) = 40\left(39{,}19 l_n\left[\frac{1}{0{,}9647\mu_0^{-3{,}833}} - 1\right] + 342\right)\cdot\left(\frac{d}{\lambda}\cdot\frac{V}{A}\right)^{0{,}77} \tag{B.4}$$

The quantity t_{fi} (μ_0) is measured in the reference tests for two values of the degree of utilisation μ_0. The Heating Rate Parameter (HRP) is unknown, but is assumed to be the same in both tests, since the tests were carried out on identical test specimens (beams & insulation systems).

In Figure B.1, a graphical presentation is given of the results of the reference tests in the load domain, together with calculated values for various values of HRP. The application range according to Equation (C.2) is also shown.

NOTE Note in this respect that - by using Equation (B.3) - condition (B.2c) is transferred from the temperature domain into the load domain, as follows:

$$0{,}45 < \mu_0 < 1{,}0 \tag{B.2d}$$



Key

Y	Fire resistance (min)
X	Relative resistance
1	HRP = 0,007
2	HRP = 0,0103
3	HRP = 0,013
4	Test data

Figure B.1 — Relation between the fire resistance and the degree of utilisation in the reference tests

B.2.3 Other features

The only "other feature" (4.5) relevant in this case is the stickability of the protection system. Two reference tests are available. During these tests, stickability failure did not occur. In addition, the relevant extrapolation amounts only 4 min (from 86 min to 90 min). Therefore it is assumed that the stickability of the fire protection does not impose any restrictions on the extended application. An extension of 4 min also meets the requirements of 4.2.1 as the increase is less than 15 min and less than 20 % of the target fire resistance of 90 min.

B.3 Model for extended application

By systemically varying the Heating Rate Parameter (HRP), a best-fit value for HRP can be determined, see Figure B.2. For the reference tests under consideration this best fit is determined as:

$$HRP_{\text{best fit}} = 0{,}0103\ [\text{m}^2\text{C/W}] \tag{B.5}$$

In general, the use of numerical tools to identify the best-fit value (e.g. method of least squares) is recommended.

Using this value of HRP, interpolation of the results of the reference tests is feasible. The limits specified in Equations B.2 (a through d) are all met.

The predicted results do not exactly match with the outcomes of the reference tests. See Table B.1, where the discrepancies are presented for the load domain. In order to compensate for these discrepancies, a modelling factor $k_{\text{m f}}$ has been introduced (4.4.2). In the given situation, this factor is calculated as follows:

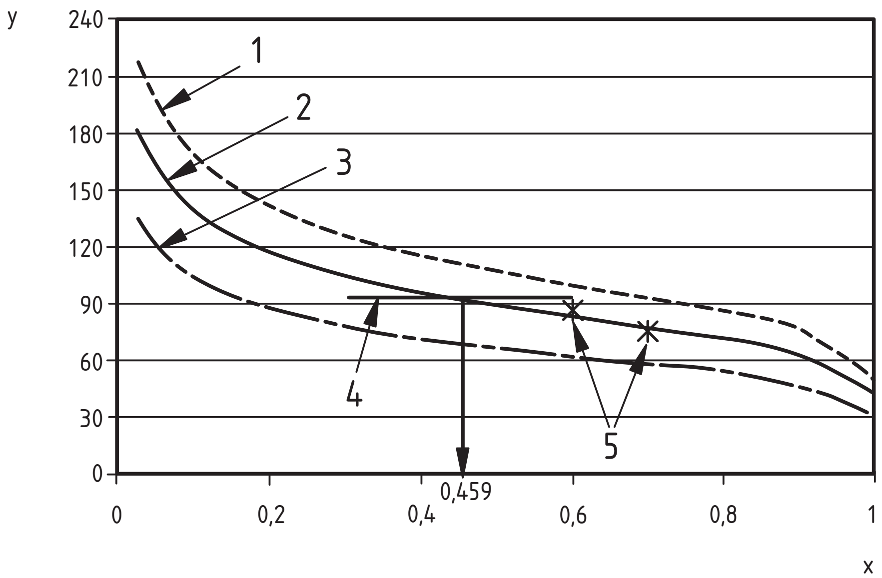
$$k_{\mathrm{mf}} = \frac{1}{2} \cdot \left(\frac{0{,}60}{0{,}54} + \frac{0{,}70}{0{,}73} \right) \Rightarrow \frac{1}{2} \cdot \left(1{,}11 + 0{,}958 \right) = 1{,}034 \tag{B.6}$$

$$k_{\mathrm{mf}} = Max\left(k_{\mathrm{mf}}, 1.0\right) = 1{,}0 \tag{B.7}$$

Hence, for a certain fire resistance, the value of utilisation predicted using this method can be accepted without any reduction.

Table B.1 — Discrepancy between the applied and predicted outcomes (load domain)

Fire resistance achieved in reference test	Degree of utilisation μ_0	
	applied in ref. test	predicted as best fit
test 1: 86 min	0,60	0,54
test 2: 75 min	0,70	0,73



Key

Y Fire resistance (mins)
X Relative resistance
1 HRP = 0,007
2 HRP = 0,0103
3 HRP = 0,013
4 Predicted for 90 minutes
5 Test data

Figure B.2 — Relation between the fire resistance and the degree of utilisation, to be used in the EXAP

The relationship between the utilisation factor μ_0 and the fire resistance, relevant for this extended application, is presented in Figure B.2. To obtain a fire resistance classification R90, the utilisation, μ_0, needs to be reduced to a value of 0,459, which is just within the field of application as mentioned in (B.2c).

NOTE Further information is available in item 5 of the Bibliography.

Annex C
(informative)

The Extended Application Of Timber Beams

C.1 Introduction

This annex gives guidance to extended application of the results of one or more reference test(s) on timber beams with or without applied fire protection.

The rules apply where reference tests have been performed in accordance with EN 1365-3 and a client may wish to do one of the following:

— increase the load-bearing capacity by increasing strength and stiffness properties by choosing timber of a higher strength class;

— increase the load-bearing capacity by increasing the dimensions of the beam;

— increase the load-bearing capacity of the beam for a smaller fire resistance time than tested;

— increase the fire resistance by applying fire protection.

It is assumed that the reference test(s) were continued until collapse of the beam was imminent at time $t_{\text{ref,fi}}$. When a reference test has been discontinued at a specified time, normally corresponding to a fire resistance rating to be achieved, and failure has not occurred, it may conservatively be assumed that failure has occurred at the time of termination of the test.

A thermal analysis is normally not necessary if the charring depth can be determined, see C.2, or the effect of charring is included in the model used, see C.2.2.

C.2 Extended application in the load domain (increase of load-bearing capacity)

C.2.1 Increasing of load-bearing capacity by higher strength class

This example applies to beams where the failure mode in the reference test is by bending failure that is failure due to lateral torsional buckling is excluded.

Where the strength class according to EN 338 or EN 1194 of the beam to be assessed is higher than that of the reference beam, the bending moment capacity of the beam to be assessed should be taken as:

$$M_1 = M_{\text{ref}} \frac{f_{\text{m,mean}}}{f_{\text{m,mean,ref}}} \tag{C.1}$$

where

M_1 is the moment resistance of the beam to be assessed;

$M_{,\text{ref}}$ is the moment resistance of the reference beam ;

$f_{m,mean}$ is the mean bending strength of the timber beam to be assessed;

$f_{m,mean,ref}$ is the mean bending strength of the timber beam in the reference test.

NOTE For mean properties of timber and LVL, see Table 1.

C.2.2 Increasing of load-bearing capacity by increasing beam dimensions (braced beams)

Where the dimensions of the beam to be assessed are increased and the charring depths of the reference beam at the end of the heating are measured and recorded, the increase of the bending moment capacity of the beam should be taken as:

$$M_1 = M_{\mathrm{ref}}\, k_{\mathrm{h}} \frac{W_{\mathrm{ef},1}}{W_{\mathrm{ef,\,ref}}} \tag{C.2}$$

where

M_1 is the bending moment capacity of the beam to be assessed;

M_{ref} is the bending moment capacity of the reference beam;

$W_{ef,ref}$ is the section modulus of the effective rectangular residual cross section of the reference beam, see below;

$W_{ef,1}$ is the section modulus of the effective rectangular residual cross section of the beam to be assessed, see below.

$$k_{\mathrm{h}} = \min \left\{ \begin{array}{c} (h_{\mathrm{ref}} / h)^{0,1} \\ 1,2 \end{array} \right\} \tag{C.3}$$

where

b_{ref} is the width of the reference beam;

h_{ref} is the depth of the reference beam;

h_1 is the depth of the beam to be assessed; for beams deeper than 600 mm, the depth should not be taken as greater than 600 mm.

The depth factor k_h given by expression (C.3) should only be applied for rectangular glued laminated beams; for other beams they may be taken equal to unity.

NOTE The factor k_h, derived from the depth factor given in EN 1995-1-1:2004 subclause 3.3, takes into account the effect of beam size on bending strength.

The section moduli $W_{\mathrm{ef,ref}}$ of the reference beam and $W_{\mathrm{ef,1}}$ of the beam to be assessed should be determined following the following steps:

1) Determine the section modulus $W_{\mathrm{ef,ref}}$ of the residual cross section from the recorded shape of the residual cross-section of the reference beam.

2) Determine the notional charring depth $d_{\mathrm{char,n,ref}}$ of the reference beam by replacing the recorded residual cross-section by a rectangular residual cross-section such that the following expression is satisfied:

3) $$W_{\mathrm{ef,ref}} = \frac{1}{6}(b_{\mathrm{ref}} - 2d_{\mathrm{char,n,ref}})\left(h_{\mathrm{ref}} - n\,d_{\mathrm{char,n,ref}}\right)^2 \quad \text{(C.4)}$$

where

n = 1 in case of charring of the sides and bottom only (3-sided fire exposure);

n = 2 in case of charring of all sides.

4) As an alternative to the procedure in step 2, the notional charring depth of the reference beam may be determined as:

$$d_{\mathrm{char,n,ref}} = \begin{cases} 1{,}23\, d_{\mathrm{char,ref}} & \text{for solid timber} \\ 1{,}08\, d_{\mathrm{char,ref}} & \text{for glued laminated timber} \end{cases} \quad \text{(C.5)}$$

where $d_{\mathrm{char,ref}}$ is the charring depth measured on the wide side of the reference beam.

Determine the section moduli $W_{\mathrm{ef,ref}}$ and $W_{\mathrm{ef,1}}$ as (see EN 1995-1-2:2004 subclause 4.2):

$$W_{\mathrm{ef,ref}} = \frac{1}{6}[b_{\mathrm{ref}} - 2(d_{\mathrm{char,n,ref}} + 7)][h_{\mathrm{ref}} - n(d_{\mathrm{char,\,n,\,ref}} + 7)]^2 \quad \text{(C.6)}$$

$$W_{\mathrm{ef,1}} = \frac{1}{6}[b_1 - 2(d_{\mathrm{char,n,ref}} + 7)][h_1 - n(d_{\mathrm{char,\,n,ref}} + 7)]^2 \quad \text{(C.7)}$$

where

b_1 is the width of the beam to be assessed;

h_1 is the depth of the beam to be assessed;

n = 1 in case of charring of the sides and bottom only (3-sided fire exposure);

n = 2 in case of charring of all sides.

C.2.3 Increasing of load-bearing capacity by decreasing the fire resistance

C.2.3.1 General

The rules given in C.2.3 should only be applied for the determination of a higher load level for a lower fire resistance time in case of one or several reference tests. There is no limitation regarding failure modes.

NOTE 1 This implies that extrapolation in the time domain is not permitted, since it cannot be excluded that other more unfavourable failure modes could occur, or charring could increase at a considerably higher rate.

The relationship η between load ratio and time to failure t_{fi} may be taken as:

$$\eta = \mathrm{e}^{-k_1 t_{\mathrm{fi}}} \tag{C.8}$$

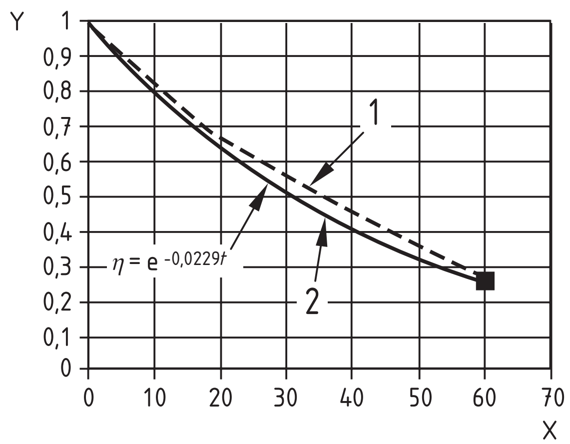
where

k_1 is a parameter;

t_{fi} is the time to failure.

NOTE 2 In Figures C.1 and C.2, a justification is given for the use of the exponential model given by expression (C.8). Figure C.1 shows the calculated relationship of load-bearing capacity and time for a glued laminated beam of dimensions 140 mm × 600 mm exposed on three sides (the upper side is assumed to be protected by a slab) according to EN 1995-1-2. The exponential model has been determined for the value at 60 min and gives conservative results.

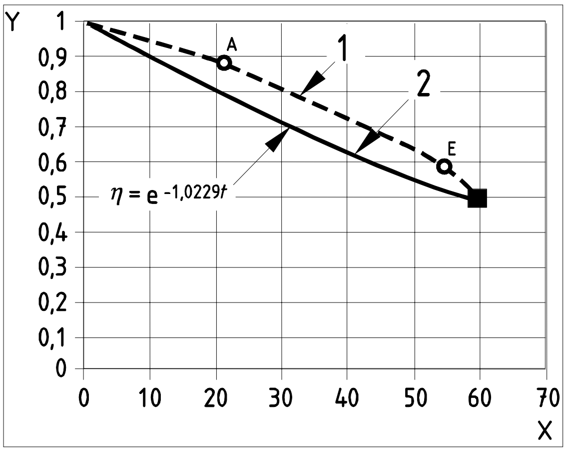
Figure C.2 shows the corresponding relationships for the same beam, clad with a 12,5 mm thick layer of gypsum plasterboard type F fixed to the beam with 41 mm long screws. Point A gives the time of start of charring, point B gives the failure time of the gypsum plasterboards due to pull-out failure of the screws. In Figure C.2 it is assumed that failure due to mechanical degradation of the gypsum plasterboard occurs later than given by point B (For gypsum plasterboard EN 1995-1-2 gives no failure times with regard to mechanical failure of the boards which must be determined by testing). For protected beams, the exponential model is more conservative.



Key

X Time (mins)
Y Relative resistance (η)
1 EN 1995-1-2
2 Exponential model

Figure C.1 — Comparison of relationships for load ratio vs. time for a glued laminated beam of dimensions 140 mm × 600 mm exposed to fire on three sides



Key

X Time (min)
Y Relative resistance (η)
1 EN 1995-1-2
2 Exponential model
A,E See notes to C.2.3.1

Figure C.2 — Comparison of relationships for load ratio vs. time for a glued laminated beam of dimensions 140 mm × 600 mm and applied protection exposed to fire on three sides

C.2.3.2 Single reference test

Where only one reference test has been performed, the coefficient k_1 should be taken as:

$$k_1 = \frac{1}{t_{\text{fi,ref}}} \ln \eta_{\text{ref}} \qquad \text{(C.9)}$$

where

$t_{\text{fi,ref}}$ is the time to failure in the reference test, in min;

η_{ref} should be taken as:

$$\eta_{\text{ref}} = \frac{M_{\text{ref}}}{M_{20}} \qquad \text{(C.10)}$$

where

M_{ref} is the bending moment in the reference beam in the fire situation;

M_{20} is the mean moment capacity of the reference beam at normal temperature (20 °C).

The mean bending moment capacity of the reference beam at normal temperature should be calculated as:

$$M_{20} = f_{\text{mean}} W_{20} \qquad \text{(C.11)}$$

where

f_{mean} is the mean value bending strength of the reference beam given in Table.1;

W_{20} is the section modulus of the original cross section in the reference test.

For a fire resistance time of $t_{fi,ref}$ with $t_{fi,1} \leq t_{fi,ref}$, the load-bearing capacity of the beam should be taken as:

$$M_1 = \eta_1 M_{20} \tag{C.12}$$

where

$$\eta_1 = e^{-k_1 t_{fi,1}} \tag{C.13}$$

$t_{fi,1}$ is the time to failure (fire resistance) of the beam to be assessed.

C.2.3.3 Several reference tests

Where more than one reference test has been performed, expression (C.12) is replaced by

$$M_1 = k_{mf}\ \eta_1 M_{20} \tag{C.14}$$

The determination of the modelling factor, see clause 4.4.2, is illustrated in the following worked example:

a) Three reference tests were conducted with three different applied loads giving rise to relative action effects $\eta_{ref,}$ see expression (C.10), and times to failure, t_{fi}, see Table C.1 and Figure C.3.

Table C.1 — Test data

	η_{ref}	t_{fi} min	η	$k_{mf,i}$	$k_{mf,I,adj}$
1	0,3$\boxed{5}$	69	0,2868	1,2203	1,2
2	0,30	62	0,3256	0,9215	0,9215
3	0,25	70	0,2817	0,8875	0,8875

b) The expression of the trendline (exponential model) is determined as:

$$\eta = \mathbf{e}^{-0,0181\mathbf{t_{fi}}} \tag{C.15}$$

c) The modelling factor $k_{mf,i}$ is calculated for each reference test *i.* All three reference tests are taken into account, since none of the values is greater than 1,4.

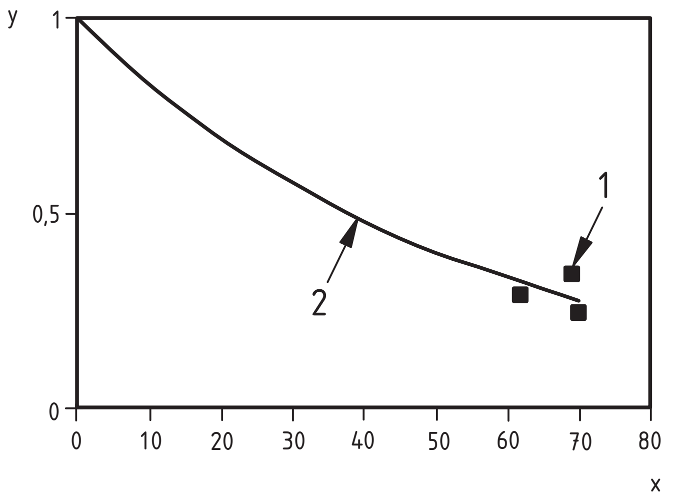
d) The modelling factors are adjusted such that no single value is greater than 1,2.

e) The total modelling factor is calculated as:

$$k_{\mathrm{mf}} = \frac{1}{3}\sum 1,2 + 0,9215 + 0,8875 = 1,0098 \qquad \text{(C.16)}$$

Since it is greater than unity, the value to be used is:

$$k_{\mathrm{mf}} = 1$$



Key

X Time (min)
Y Relative resistance
1 Reference tests
2 Exponential model

Figure C.3 — Reference test results and exponential model

C.3 Extended application in the time domain: Increasing fire resistance by applied fire protection

The prerequisite of the following example is that a beam without applied protection has been tested in a reference test. The charring depth of the reference beam at the end of the heating has been measured and recorded. The task is to increase the fire resistance by fixing fire protective cladding made of 12,5 mm thick gypsum plasterboard type A in accordance with EN 520 to the beam.

The extended application should include the following steps:

1) Determine the notional charring depth of the reference beam, $d_{\mathrm{char,n,ref}}$, as shown in C.2.2;

2) Determine the notional charring rate of the reference beam, $\beta_{\mathrm{n,ref}}$, as:

3) $$\beta_{\mathrm{n,ref}} = \frac{d_{\mathrm{char,n,ref}}}{t_{\mathrm{fi,ref}}} \qquad \text{(C.17)}$$

where $t_{\mathrm{fi,ref}}$ is the time to failure in the reference test.

4) P
ut $d_{\mathrm{char,n,1}} = d_{\mathrm{char,n,ref}}$ (C.18)

5) Determine the time of start of charring, t_{ch}, of the beam as (see EN 1995-1-2:2004 clause 3.4.3.3):

6) $t_{ch} = 2,8\, h_p - 14$ (C.19)

where h_p is the thickness of the cladding, in mm.

7) Determine the notional charring rate of the beam to be assessed, $\beta_{n,1}$, as (see EN 1995-1-2:2004, clause 3.4.3.2):

8) $\beta_{n,1} = k_3\, \beta_n$ (C.20)

where

$k_3 = 2$ for $d_{char,n} \leq 25$ mm ;

$k_3 = 1$ for $d_{char,n} > 25$ mm .

9) Determine the fire resistance of the beam to be assessed as:

$$t_{fi,1} = t_{ch} + \frac{d_{char,n,1}}{\beta_{n,1}} \quad \text{for } d_{char,n,1} \leq 25 \text{ mm} \tag{C.21}$$

and, taking account of expression (C.22),

$$t_{fi,1} = t_{ch} + \frac{25}{2\,\beta_n} + \frac{d_{char,n,1} - 25}{\beta_n} \quad \text{for } d_{char,n,1} > 25 \text{ mm} \tag{C.22}$$

For applied protection made of other materials or materials for which no failure time of the protection is given in EN 1995-1-2, t_{ch} should be determined from tests according to ENV 13381-7. For applied fire protection remaining in place after the start of charring of the beam, a reduced notional charring rate $k_2\, \beta_n$ should be determined which is valid until failure time t_f of the protection, with k_2 and t_f either taken from EN 1995-1-2:2004, clauses 3.4.3.2 and 3.4.3.4, or determined by testing according to ENV 13381-7.

Annex D
(informative)

The Extended Application of a Composite Steel Concrete Beam

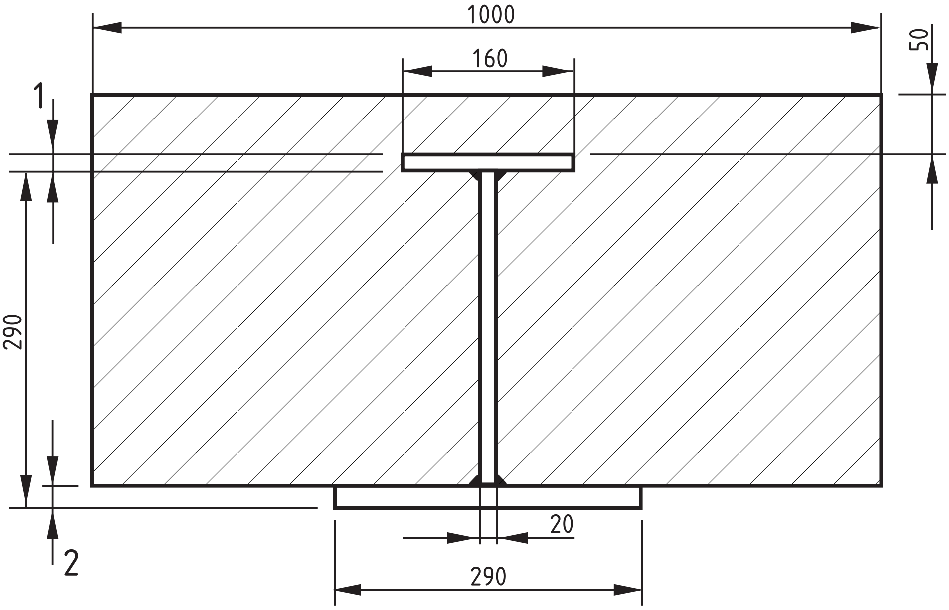
D.1 Introduction

D.1.1 General

This example in this annex illustrates a beam made of two different materials (steel and concrete) and shows how the calibration adjustments for the thermal analysis and structural analysis are applied.

Two loaded beam fire tests, the reference tests, have taken place on beams of slightly different sizes (Figure D.1). The unprotected steel beams are constructed from three welded plates and are embedded into the concrete floor slab. Nominal reinforcement spans over the top flanges in the 50 mm deep concrete topping.

All dimensions in millimetres



Key

1	Top flange	Test 1	15 mm	Test 2	12 mm
2	Bottom flange	Test 1	25 mm	Test 2	18 mm

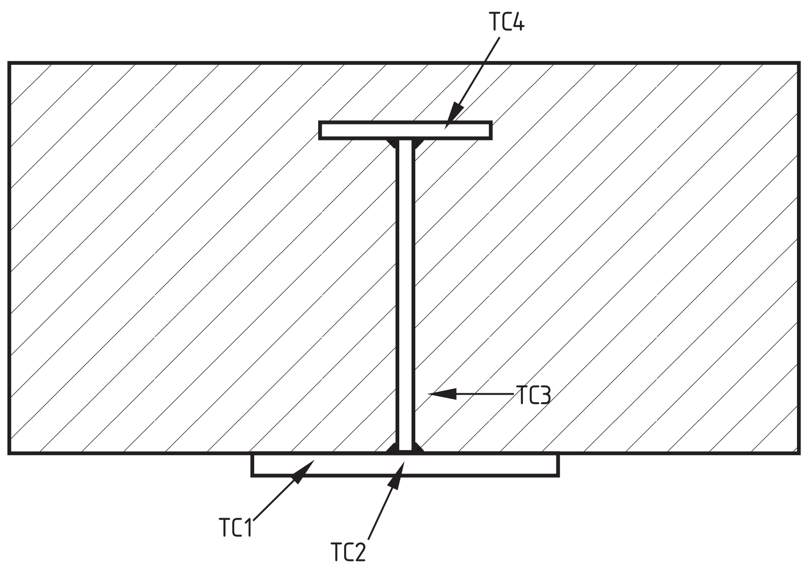
Figure D.1 — Cross-section through test specimens

D.1.2 Reference test 1

Span	4,2 m
Steel grade	S275
Steel strength	Not measured
Concrete grade	30 N/mm^2
Concrete strength	Not measured
Applied loading	four loads of 120 kN
Self weight	30 kN
Fire resistance	69 min
Mode of failure	Overall bending

D.1.3 Reference test 2

Span	4.2 m
Steel grade	S275
Steel strength	Not measured
Concrete grade	30 N/mm^2
Concrete strength	Not measured
Applied loading	four loads of 115 kN
Self weight	30 kN
Fire resistance	66 min
Mode of failure	Overall bending



Key

TC1 Position of Thermocouple 1
TC2 Position of Thermocouple 2
TC3 Position of Thermocouple 3
TC4 Position of Thermocouple 4

Figure D.2 — Thermocouple numbers and positions

The client requires a beam to carry normal floor loading of 5 kN/m^2 with a span of 10 m and a spacing to the next beams of 6 m. The required fire resistance is 60 min.

D.2 Analysis of reference tests

D.2.1 Thermal performance

D.2.2 Reference test 1

The measured temperatures at failure and at 60 min. The mean temperature is shown. The thermocouple numbers and positions are shown in Figure D.2. No temperatures were measured in the concrete.

Table D.1 — Measured temperatures in reference test 1

Thermocouple	Temperature at 60 min (°C)	Temperature at 69 min (°C)
TC1 (flange 0,25 point)	735	778
TC2	660	702
TC3 (30 mm up from top of flange	452	480
TC4	61	67

The temperature distribution in the reference test has been predicted using a finite difference thermal analysis program. The program has been shown to predict temperatures reasonably well in similar beams and meets the criteria for advanced calculation models given in EN 1994-1-2. For the purpose of analysis, the beam is divided into a number of rectangular elements and the program computes the heat flow between adjacent elements and short time steps. The predictions and measured temperatures are compared in Table D.2.

Table D.2 — Comparison of measured and predicted temperatures in reference test 1

Thermocouple	Temperature at 60 min (°C)		Temperature at 69 min (°C)	
	Measured	**Predicted**	**Measured**	**Predicted**
TC1	735	722	778	760
TC2	660	634	702	671
Weighted mean	710	698	753	736
TC3	452	434	480	462
TC4	61	57	67	75
NOTE The measured weighted mean = (2 × TC1 + TC2)/3 The predicted weighted mean is the mean of 5 elements used in the thermal modelling.				

The predicted temperatures are close to the measured values but the program appears to predict a low temperature for TC2, at the centre of the bottom flange. The mean bottom flange temperature is predicted quite well. The prediction of TC3 is unconservative.

On the basis of Reference Test 1, some modification to the predicted temperatures may be required when predicting other cases for 60 min fire resistance.

D.2.3 Reference test 2

D.2.3.1 General

The measured temperatures at failure and at 60 min are shown in Table D.3. No temperatures were measured in the concrete.

Table D.3 — Measured temperature in reference test 2

Thermocouple	Temperature at 60 min (°C)	Temperature at 66 min (°C)
TC1	750	783
TC2	658	682
TC3	460	472
TC4	65	82

The predictions and measured temperatures are compared in Table D.4.

Table D.4 — Comparison of measured and predicted temperatures in reference test 2

Thermocouple	Temperature at 60 min (°C)		Temperature at 69 min (°C)	
	Measured	**Predicted**	**Measured**	**Predicted**
TC1	750	738	783	775
TC2	658	632	682	656
Weighted mean	719	713	749	743
TC3	460	443	472	463
TC4	65	63	82	75

The predicted temperatures are again close to the measured values but the program appears to predict a low temperature for TC2, at the centre of the bottom flange. The mean bottom flange temperature is predicted quite well. The prediction of TC3 is unconservative.

Conclusion of thermal performance

On the basis of the above comparisons, the thermal analysis program can be used to model this type of composite beam but some small calibration corrections should be applied to the results.

D.2.3.2 Correction to bottom flange

The difference between the measured and predicted temperatures was greatest in Reference Test 1 and for both tests the prediction was unconservative. For 60 min, therefore, any predicted temperatures should be increased by 12 °C (the difference in weighted mean temperatures). At about 700 °C, this correction could affect the flange strength by 6 %.

D.2.3.3 Correction to lower web

The difference between the measured and predicted temperatures was at greatest 18 °C in Reference Test 1 and 17°C in Test 2. For 60 min, therefore, any predicted temperatures should be increased by 18°C.

D.2.4 Structural performance

Eurocode 4, EN 1994-1-2, does not cover this exact type of structure although it gives methods of computing the moment resistance of composite sections.

A beam of this type is torsionally stable and can only fail by bending or shear. EN 1994-1-2 states that, for beams of this type, no check of shear resistance in fire is necessary.

D.2.5 Bending resistance

The applied bending moment in Reference Test 1 was 267,8 kNm and in Reference Test 2 was 257,3 kNm. No material strengths were measured. In accordance with 4.4.3, mean properties of steel and concrete should be used in assessing the Reference Tests and in any predictions. Material strengths at elevated temperatures are taken from EN 1994-1-2.

Assume that for S275 steel the mean strength is 110 % of the characteristic value i.e. 302,5 N/mm^2 ($275 \times 1,1$) and that for concrete the mean strength is 38 (30 + 8).

NOTE In this example, for simplicity, the shear connection between the concrete and steel is not considered. A full analysis should include this effect.

D.2.6 Assessment of reference test 1

The structural model used is a simple plastic analysis of the cross-section as specified in EN 1994-1-2. The cross-section is divided into 5 elements:

1) Concrete above top flange.

2) Top flange of steel web of welded steel section.

3) Upper 150 mm of steel web of welded steel section.

4) Lower 70 mm of web of welded steel section.

5) Bottom flange of welded steel section.

A material strength and temperature are assigned to each element. The plastic moment resistance may be calculated by hand but, here, Excel based calculations are presented in Table D.5 and Table D.6.

Table D.5 — Calculation of moment resistance in reference test 1

Width (mm)	Depth (mm)	Material	Cold strength (N/mm²)	Temp (°C)	Centroid (mm)	Hot strength (N/mm²)	Force (kN)	Moment contribution (kNm)
1000	50	Concrete	38	20	25,0	33,0	1900,0	-47,5
160	15	Steel	302,5	67	57,5	302,5	--	Element split
160	4,6	Steel	302,5	67	53,6	302,5	348,7	-11,7
160	10.4	Steel	302,5	67	61,1	302,5	377,3	30,0
20	150	Steel	302,5	300	140,0	302,5	907,5	127,1
20	70	Steel	302,5	480	250,0	249,3	349,0	87,2
290	25	Steel	302,5	753	297,5	50,3	364,9	108,6
				Moment resistance (kNm)				293,7
				Compressive force in concrete (kN)				1 900,0
				Compressive force in steel (kN)				223,7
				Tensile force in steel (kN)				2 123,7

NOTE The plastic neutral axis is in the top flange of the welded steel section. This element is split into 2 elements with the upper part being in compression and the lower part in tension.

The calculated moment resistance is 293,7 kNm. This is 9,7 % greater than the applied moment in the test (267,8 kNm). This difference is reasonable for this type of calculation.

D.2.7 Assessment of reference test 2

Table D.6 — Calculation of moment resistance in reference test 2

Width (mm)	Depth (mm)	Material	Cold strength (N/mm²)	Temp (°C)	Centroid (mm)	Hot strength (N/mm²)	Force (kN)	Moment contribution (kNm)
1000	50	Concrete	38	20	25,0	33,0	1650,0	-41,3
160	12	Steel	302,5	82	56,0	302,5	--	Element split
160	2,8	Steel	302,5	82	52,7	302,5	262,8	-7,1
160	9,2	Steel	302,5	82	58,7	302,5	318,0	25,4
20	160	Steel	302,5	300	142,0	302,5	968,0	137,5
20	70	Steel	302,5	472	257,0	254,6	356,4	91,6
290	18	Steel	302,5	749	301,0	51,8	270,3	81,4
				Moment resistance (kNm)				281.3
				Compressive force in concrete (kN)				1900,0
				Compressive force in steel (kN)				137,8
				Tensile force in steel (kN)				2 037,8

NOTE The plastic neutral axis is in the top flange of the welded steel section. This element is split into 2 elements with the upper part being in compression and the lower part in tension.

The calculated moment resistance is 281,3 kNm. This is 14 % greater than the applied moment in the test (246,8 kNm).

D.2.8 Conclusions of structural performance

The EN 1994-1-2 method of calculating the moment resistance predicted strengths 9,7 % and 14 % higher than measured in the tests. These differences could be explained by errors in the assumed material properties, insufficient temperature data or could be due to some other unidentified cause.

The modelling factor (see 4.4.2) (k_{mf}) is therefore:

$$k_{mf} = \frac{1}{2}\left(\frac{1}{1{,}097} + \frac{1}{1{,}14}\right) = 0{,}894$$

NOTE As the ratio F/R for each test is less than 1,2, the actual values do not have to be limited to 1,2.

D.2.9 Model for extended application

The thermal and structural models used to analyse the reference tests may be used with the following corrections applied. On the basis of the data presented in this example any extended application can only be for 60 min.

Temperatures	Bottom plate	+12 °C
	Lower web	+18 °C
Strength	M_{f}	0,894

D.2.10 Extended application

D.2.10.1 General

It is assumed that, for normal conditions, the design conforms to EN 1994-1-2.

D.2.10.2 Calculation of load

Floor slab	3 kN/m^2
Imposed load	5 kN/m^2
Weight of steel	0,2 kN/m^2

The loads are factored using factors from EN 1991-1:

Factored bending moment = $[(3 + 0{,}2) \times 1{,}0 + 5 \times 0{,}5] \times 10^2 \times 6 / 8$ = 427,5 kNm

It is convenient to adjust the required bending resistance to take account of the Modelling factor. Using the plastic bending resistance model, the bending resistance should be 478,2 (= 427,5/0,894).

Following the guidance in A.7, no calculation of deflection is required.

The proposed beam cross-section is shown in Figure D.3. The section size has been made deeper and the web thickness has been increased compared with the reference tests. The slab width of 2 500 mm is based on 25 % of the 10 m span (EN 1994-1-1).

The predicted temperatures and corrected temperatures are shown in Table D.7 and the calculation of moment resistance is shown in Table D.8.

The calculated bending resistance is 503,9 kN so the design is adequate, based on the extended application analysis.

All dimensions in millimetres

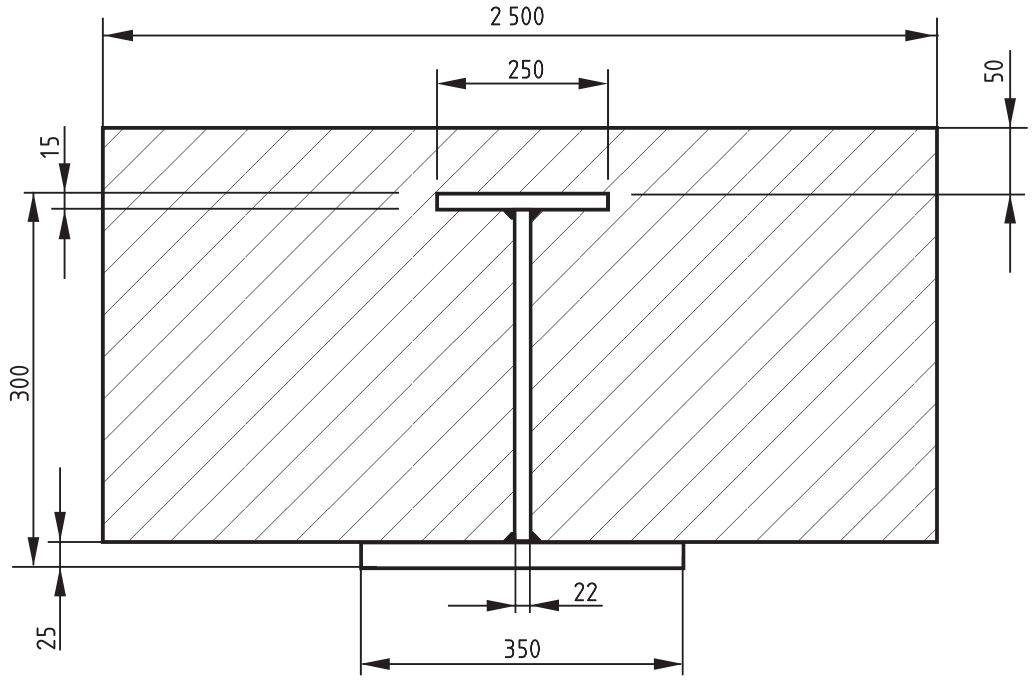


Figure D.3 — Cross-section through required design

Table D.7 — Predicted temperatures and corrected temperatures

Position (Thermocouple)	Predicted temperatures at 60 min (°C)	Corrected temperatures (°C)
TC1	722	
TC2	731	
Weighted mean	699	699 + 12 = 711
TC3	470	470 + 18 = 488
TC4	56	

Table D.8 — Calculation of moment resistance in design example

Width (mm)	Depth (mm)	Material	Cold strength (N/mm²)	Temp (°C)	Centroid (mm)	Hot strength (N/mm²)	Force (kN)	Moment contribution (kNm)
2500	50	Concrete	38	20	25,0	38,0	--	Element split
2500	35	Concrete	38	20	17,5	38,0	3328,4	-58,3
250	15	Steel	302,5	56	57,5	302,5	1134,4	65,2
22	187	Steel	302,5	300	158,5	302,5	1244,5	197,3
22	70	Steel	302,5	488	287,0	243,9	375,7	107,8
350	25	Steel	302,5	711	334,5	65,6	573,8	192,0
2500	50	Concrete	38	20	25,0	38,0	--	Element split
				Moment resistance (kNm)				503,9
				Compressive force in concrete (kN)				3 328,4
				Compressive force in steel (kN)				zero
				Tensile force in steel (kN)				3 328,4

NOTE The plastic neutral axis is in the top flange of the welded steel section. This element is split into two elements with the upper part being in compression and the lower part in tension.

Annex E
(informative)

The extended application of concrete beams

E.1 Introduction

This annex gives some examples for extended application of the results of one or more reference test(s) concrete beams.

In most cases extended application of concrete beams can be carried out by using design rules of EN 1992-1-2.

E.2 Failure modes

Failure modes of reinforced or prestressed concrete beams exposed to fire may be:

a) Tension failure of the reinforcement in bending.

   This is the most common type of failure and can be predicted by rapid increase of deflection.

b) Compression failure of concrete in bending.

   Possible, if the compression side of the cross section is exposed to fire, like continuous beams near the supports, or thin beams.

c) Shear failure.

   Possible for beams with thin webs, like I-beams.

d) Anchorage failure.

   Could be possible for prestressed beams, but normally prevented by stirrups in the anchorage zone.

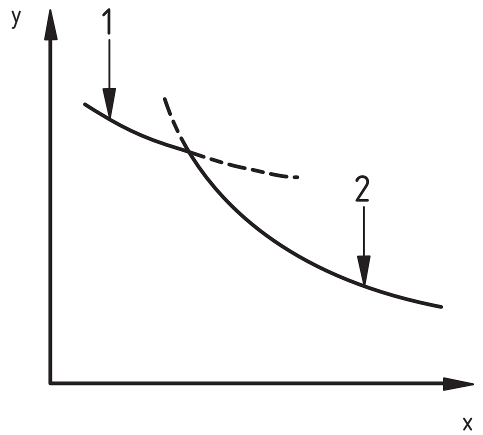
e) Spalling.

   EN 1992-1-2 includes design rules to prevent spalling.

E.3 Examples

E.3.1 Possible change of failure mode

A typical resistance curve of a concrete beam (constant cross section) is shown in Figure N.1. If there is bending failure in fire test, the results can be extended to longer spans without risk for shear failure.



Key

X Span (m)
Y Load (kN/m)
1 Shear
2 Bending

Figure E.1 — Typical resistance curve of a concrete beam

E.3.2 Change of cross section

E.3.2.1 Simple rule for bending

If the average axis distance of the main reinforcement does not change. Cross section dimensions, load, span and amount of reinforcement may be changed when it is verified by calculation that the tensile stress in the reinforcement does not increase.

NOTE When the load level in the test has been chosen to give maximum steel stress, this verification is automatically covered by normal temperature design.

However, decrease of the width may increase temperature in the critical areas of the cross section, and should not be done without more detailed calculations.

E.3.2.2 Simple rule for shear

Ratio $V_{fi}/f_{cd}bd$ should not increase and average axis distance of stirrups should not decrease.

E.3.3 Change of material strength

a) Strength class of concrete.

Test results are valid for all classes of normal strength concrete (≤ C50/60) with same type of aggregates (siliceous, calcareous or lightweight) assuming that load level is kept unchanged.

b) Type of reinforcement: reinforcing steel, prestressing bar, wire or strand.

Result can not be used for other types of reinforcement unless the influence of differences in thermal and mechanical properties is considered, see e.g. ENV 1992-1-2.

c) Strength of reinforcing or prestressing steel.

Change of strength has no influence on fire resistance assuming that load level is kept unchanged.

E.3.4 Axial and rotational restraint

Concrete beams are normally tested as simply supported. These results are on the safe side for the other cases because axial and rotational restraint increases the fire resistance.

NOTE Axial restraint may decrease fire resistance if it is situated above the centroid axis of the beam. This should be prevented by design rules.

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