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On the Fire Resistance of Structural Steel Elements Derived from Standard Fire Tests or by Calculation

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SUMMARY

Due to high costs, a fire resistance test of a load-bearing structural element is usually limited to one test specimen — in a few countries, to two test specimens. Accordingly, there are no possibilities of evaluating the test results statistically.

For a single test specimen, the actual quality of the structural material represents a random sample from a wide variety. This applies also to the initial imperfections of the structural elements. In consequence of this, a standard fire resistance test is generally carried out on a test specimen with a load-bearing capacity which is greater — most often significantly greater — than the load-bearing capacity related to the characteristic values of the mechanical material strength and of the imperfections of the structural member. In current practice, no corrections of the test results with respect to this are made.

In a conventional analytical design, a determination of the load-bearing capacity of a structure at room temperature conditions is based on the characteristic values of the strength and imperfections. Extended to a structural fire engineering design, this procedure will give an analytically determined fire resistance of a load-bearing structural element which is lower — normally essentially lower — than the corresponding value derived from a standard fire resistance test.

Available methods for a simplified calculation of the temperature of fire exposed steel structures are, as a rule, based on the assumption of a uniformly distributed temperature over the cross section and the length of the structure at each time of fire exposure. The ECCS Recommendations for an analytical design of steel structures exposed to a standard fire follow this kind of approach. For certain types of steel structures, for example, beams with a slab on the upper flange, a considerable temperature variation arises over the cross section as well as in the longitudinal direction during a fire resistance test. A simplified, analytical method, which neglects this influence, gives a further underestimation of the fire resistance in relation to the corresponding result obtained in a standard fire resistance test.

The described discrepancies between an analytical and an experimental determination of the fire resistance are further discussed and analysed in Sections 2 and 3, with particular reference to different types of steel structures. The discussion is focussed on the loading and restraint conditions, the scatter of material properties and geometrical imperfections, and the temperature variation over the structure or structural element. The discussion is summarized in Section 4 and alternative methods of correction are outlined briefly for obtaining an improved consistency between the analytical and the experimental approaches.

In Section 5, one of these methods is further developed to a design basis which can be applied easily in practice. Principally, the method is characterized by a correction of the analytically determined load-bearing capacity, based on the characteristic value of the structural material properties, the characteristic value of the imperfections of the structure, and a uniformly distributed steel temperature across and along the structure. Two different

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sequences of the design procedure are dealt with, defined according to Figs. 10 and 11. The resultant correction factors, f and κ, belonging to the respective sequences, are given by Figs. 8 and 12 for columns, isostatic beams, and hyperstatic beams. The straight line curves in Figs. 9 and 13 show corresponding, simplified relationships for the f and κ factors.

The derived method of correction must be characterized as an approximate approach. This is in consequence of the present state of knowledge, which does not allow a solution of high accuracy. The task to develop a correction procedure which leads to improved consistency between an analytically and an experimentally determined fire resistance, should also be seen in the context of the inadequate reproducibility of the standard fire resistance test.

1. DEFINITION OF THE PROBLEM

During the last ten years important progress can be noted in the development of computation methods for an analytical determination of the behaviour and load-bearing capacity of building structures and structural elements at fire exposure. For steel structures, this development now has arrived at a point which enables an analytical fire engineering design to be carried out in most practical applications. Consequently, more and more countries are now permitting a classification of load-bearing structures with respect to fire to be formulated analytically. The European recommendations for the design of fire-exposed steel structures exposed to the standard fire, recently drawn up by the European Convention for Constructional Steelwork [1], will certainly stimulate more countries officially to accept this means of classification as an alternative to the internationally prevalent method of classification based on results of standard fire resistance tests.

A direct application of ordinary assumptions in an analytical fire classification of steel structures (characteristic values of the mechanical properties of the material, characteristic values of the geometrical imperfections of the structure, uniform temperature distribution across and along the structure), however, generally results in a calculated level of fire resistance which is lower — normally essentially

lower — than the corresponding level, measured in a standard fire resistance test. This paper deals with the problem of such a systematic discrepancy and develops a method for avoiding this and arriving at more consistent results in the analytical and experimental approaches.

2. STANDARD FIRE RESISTANCE TESTS

Fire resistance tests of load-bearing structural elements and partitions have been performed frequently for more than half a century. The tests are carried out according to national specifications which may have minor variations of detail from country to country. In principle, the test conditions and the associated performance criteria are internationally harmonized in conformity with ISO standard 834 [2].

The fire resistance of a test specimen is defined as the time, expressed in minutes, of the duration of a specified heating until failure occurs under conditions — load-bearing capacity, insulation, integrity — appropriate to the specimen. In the test, the specimen is exposed in a furnace to a temperature rise, which shall be controlled so as to vary with time within given limits according to the relationship:

$$T - T_o = 345 \log_{10} (8t + 1) \qquad (2.1)$$

where t = time, in minutes, T = furnace temperature at time t, in °C, T_o = furnace temperature at time t = 0, in °C.

For steel structures, ordinarily only the criterion of load-bearing capacity has relevance. In reporting the test results, the time of fire resistance then, as a rule, is supplemented by information on the so-called critical temperature of the test specimen, which is defined as the measured maximum steel temperature at failure of the specimen.

Internationally, the standard fire resistance test according to ISO 834 is considered to be one of the fire test methods most thoroughly dealt with. The frequent use of the test has given important and extensive information on the fire performance of various types of building structural elements. Without this information, the progress in developing analytical design methods for fire exposed structures and structural elements would hardly have been

possible. Despite this, the standard fire resistance test can be criticized seriously. The specification of the test procedure is insufficient in several respects, for instance, as concerns the heating characteristics, the environment of the furnace, and the thermocouples for measuring and regulating the furnace temperature. As a consequence, the fire resistance of one and the same structural element can vary considerably when the element is tested in different fire engineering laboratories with varying furnace characteristics, compare, for instance, refs. 3 and 4. An improvement in the reproducibility of the fire resistance test in this respect has a high degree of priority. The problem does not have any consequences on an analytical fire classification.

In addition to the furnace characteristics, the following five factors are particularly important in a comparative discussion of an analytical and an experimental determination of the fire resistance of load-bearing structural elements.

2.1. Loading of the structural element

According to ISO 834, the test specimen shall be subjected to a loading which, in the critical regions of the element, produces stresses of the same magnitude as would be produced normally in the full-size element when subjected to the design load (usually, the load corresponding to maximum permissible stress). The design load specifications may differ from country to country. Although this complicates a comparison of the results of tests performed in different countries on the same structural element, it does not give rise to problems when comparing test results with analytical results.

Modern design philosophy, characterized by the concept "design load effect" (based on characteristic load values in combination with partial factors and load combination factors) and the concept "design strength" (based on characteristic strength in combination with partial factors with respect to scatter in material strength, uncertainty of design model, safety class), calls for an international discussion on how the fire test load logically should be chosen if the test results are intended for practical application within such a design system. However, this is outside the scope of the present paper.

2.2. Material properties

The fire resistance of a load-bearing structural element depends decisively on the mechanical properties of the structural materials at elevated temperatures. These properties are related to the actual material quality at ordinary room temperature.

Fire resistance tests are very expensive and, consequently, the number of tests on each prototype of a structural element ordinarily is limited to only one test — in a few countries, two tests. Hence, there are no possibilities of evaluating the test results statistically.

The actual quality of the structural material in a single test specimen can be considered as a random sample from a wide variety. Therefore, the material quality of a structural element used in a fire resistance test will generally be higher than the quality guaranteed by the manufacturer and, consequently, the mechanical properties of the material will be better than the characteristic values.

In the commentary to the standard ISO 834 it is recommended that, if possible, the test load in a fire resistance test be related to the ultimate load of the test element before heating. In present fire testing practice, however, neither the test load nor the test results are adjusted with respect to the difference between the actual random value and the characteristic value of the material strength.

The phenomenon may be illustrated using, as an example, a steel beam of Fe E 240 material. This quality has a characteristic yield stress value of 240 MPa at ordinary room temperature. In many cases, however, a steel beam made of this material can have a real yield stress as high as $\cong 300$ MPa. Exposed to the same design load, a steel beam with a yield stress of 300 MPa will collapse in a fire resistance test at a steel temperature which is about 75 °C higher than the corresponding collapse temperature for a steel beam with a yield stress of 240 MPa.

2.3. Geometrical imperfections and residual stresses

The structural behaviour of an axially compressed steel column is considerably influenced by initial geometrical imperfections and residual stresses. This applies to room temperature as well as elevated temperature conditions. As for the mechanical properties of the structural material, the real level of this imper-
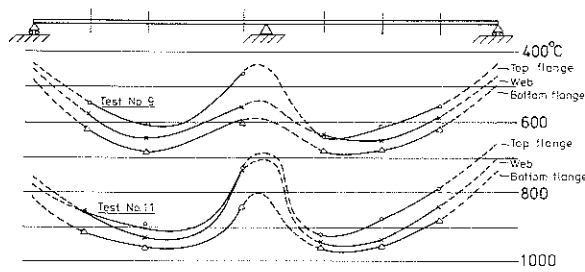
Fig. 1. Measured temperature distribution along span and over cross section for two tests of fire exposed, continuous steel I beams. Test No. 9: steel beam, insulated by 15 mm vermiculite plaster; test No. 11: non-insulated steel beam [5].

fection influence for a single test specimen can also be considered as a random sample from a wide variety. Consequently, a steel column element used in a fire resistance test generally has a lower level of imperfection than the characteristic level specified in codes and regulations. No correction of the test results is made in current fire testing practice in respect of this.

2.4. Temperature distribution across and along the structural element

Depending on the heating characteristics of the furnace, a beam or a column will have a considerable temperature variation in its longitudinal direction in a fire resistance test. For beams with a roof or floor slab on the upper flange, a considerable temperature variation over the cross section is added. As a single example, Fig. 1 shows the measured temperature distribution along the span and over the height of the cross section for two fire resis-

tance tests of steel I beams, continuous in two spans [5].

2.5. Restraint of the structural element

In buildings, the deformation of the structural elements due to heating from fire exposure is quite often partly restrained by connecting building components. Two main types of restraint can then occur — axial and rotational.

The Standard, ISO 834, specifies that a test specimen shall be supported and restrained at its ends or sides, in a fire resistance test, in a way which is as far as possible similar in nature to that which is valid for the corresponding structural element in service. Several fire engineering laboratories also have facilities to produce end restraints but very few laboratories have facilities for measuring the real degree of restraint in a well-defined manner. Consequently, the degree of restraint of the test specimen is quite often unknown in a fire resistance test and it may also vary greatly during the test. This makes comparisons of test results from different fire engineering laboratories — as well as a practical interpretation of the test results — difficult. The problem seems to be most manifest for slender columns.

3. CHARACTERISTICS OF AN ANALYTICAL DETERMINATION OF THE FIRE RESISTANCE OF STEEL STRUCTURES ACCORDING TO ECCS RECOMMENDATIONS

An analytical determination of the fire resistance of load-bearing structures and structural elements follows a design procedure according to Fig. 2.
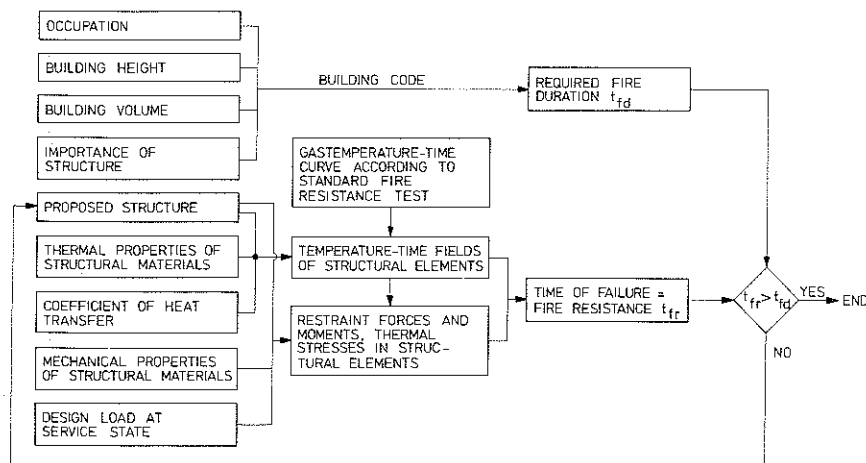

Fig. 2. Procedure for an analytical fire engineering design of load-bearing structures, based on classification and thermal exposure, according to the standard fire resistance test.

For different applications, the codes and regulations give the required time of fire duration, t_{fd}, for which the structure has to fulfil its load-bearing function. The required fire duration time then ordinarily depends on the occupation, the height and volume of the building, and the importance of the structure or structural element.

The theoretical determination of the fire resistance of the structure, t_{fr}, is based on the gas temperature–time curve, specified for the standard fire resistance test — eqn. (2.1.). With this information as input, the temperature–time fields of the fire-exposed structure or structural element can be calculated, using

(1) the structural characteristics;

(2) the thermal properties of the structural materials;

(3) the coefficients of heat transfer for the various surfaces of the structure

as further input data. For fire-exposed steel structures, this calculation can be performed comparatively quickly by the application of the design basis in the European recommendations [1]. Introducing

(4) the mechanical properties of the structural materials;

(5) the load characteristics,

then the time variation of the restraint forces and moments, the thermal stresses and the load-bearing capacity can be computed. The time at which the load-bearing capacity has decreased to the level of the design load at service state defines the time of failure or the fire resistance, t_{fr}, of the structure. The design criterion to be satisfied is, that $t_{fr} > t_{fd}$.

For steel structures, the analytical transfer of a temperature field to a load-bearing capacity can be carried out simply by a limit state design according to the elementary plastic theory in those cases when a similar design is allowed at ordinary room temperature. The ultimate load will then be based on a temperature-dependent effective yield stress, σ_{yT}. Alternatively, the load-bearing capacity can be determined by computing the deflection curve of the fire exposed steel structure and defining the ultimate state by a criterion with regard to a limit deflection or a limit rate of deflection [6]. In ref. 1, stress–strain relationships are given for various grades of steel — Fe 310, Fe 360, Fe 430 and Fe 510 — at elevated temperatures up to 600 °C. The relationships include the approximate influence of
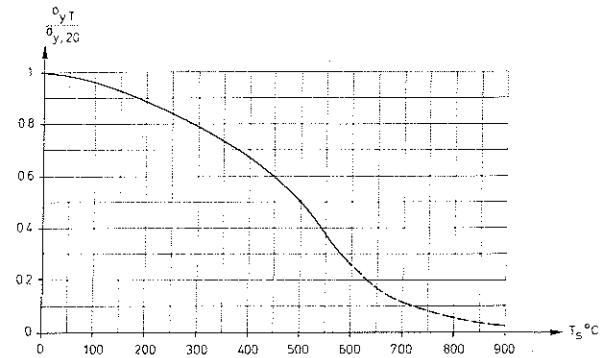

Fig. 3. Quotient between effective yield stress at elevated temperature, σ_{yT}, and yield stress at ordinary room temperature, $\sigma_{y,20}$, as a function of steel temperature, T_s [1].

high temperature creep. The effective yield stress, σ_{yT}, derived from these relationships as a function of the steel temperature T_s, is reproduced in Fig. 3.

In Section 2, five factors have been commented on which are of particular importance to the analytical and experimental approaches in a determination of the fire resistance of load-bearing structural elements. As far as the European recommendations for the design of fire exposed steel structures are concerned, these five factors have the characteristics given below.

3.1. Loading of the structural element

In conformity with ISO standard 834 for fire resistance tests, the load level is assumed to be equal to the design load also in the analytical approach according to the ECCS Recommendations [1].

3.2. Material properties

The stress–strain relationships and the connected effective yield stress, σ_{yT}, as given in the ECCS Recommendations [1], are based on the characteristic values of the mechanical properties for various grades of steel at elevated temperatures. These values can be considered to coincide with the 2.3% confidence level, which is in accordance with the fundamental principles of a structural design for ordinary room temperature application.

In contrast to this, the actual quality of the structural material in a single specimen used in a fire resistance test represents a random sample from a wide variety — as pointed out in more detail in Section 2.2.

Extensive investigations have been reported regarding the yield stress scatter of mild structural steels at ordinary room temperature. From these investigations — cf., for instance refs. 7 and 8 — the value

$$\frac{\bar{M}}{\bar{M} - 2\sigma_M} = 1.2 \qquad (3.2.1)$$

can be derived as representative of the quotient between the mean value, $\bar{M}$, and the characteristic value $\bar{M} - 2\sigma_M$ of the yield stress. σ_M is the standard deviation. The same value of the quotient can also be considered as roughly representative of the yield stress at elevated temperatures.

3.3. Geometrical imperfections and residual stresses

In the structural design of steel columns, the influence of initial deflections, unintentional eccentricity and residual stresses can be taken into account in an integrated way by a parameter of imperfections. This parameter can be formulated as a hypothetical initial deflection, a hypothetical eccentricity or a combination of these two quantities. In the European recommendations for the design and construction of steel structures (ECCS 1977), the parameter of imperfection is chosen as a hypothetical eccentricity in such a way that the failure load at ordinary room temperature coincides with the 2.3% confidence level.

3.4. Temperature distribution across and along the structural element

In Section 2.4, the temperature variation, obtained in a fire resistance test, across and along a steel beam with a roof or floor slab on the upper flange was described and illustrated fragmentarily (see Fig. 1). For a steel column thermally exposed on all sides, the temperature variation over the cross section is ordinarily negligible, while a not inconsiderable temperature variation generally arises along the test specimen in a fire resistance test.

Analytical methods exist which enable an accurate determination of the temperature variation over the cross section of a fire exposed steel beam in various structural applications. Such a method is presented in ref. 9, together with computer routines. The algorithm described can easily be coupled to most finite element programs. An illustration of the capability of the theory is given in Fig. 4,

which shows the calculated temperature distribution along the line of symmetry of a gypsum-insulated steel beam with a concrete slab on the top flange, at selected times of thermal exposure according to the standard fire resistance test, eqn. (2.1).

A temperature variation along a fire exposed structure or structural element can easily be taken into account in an analytical design, if the corresponding thermal exposure characteristics can be specified. The present state of knowledge, however, does not allow this to be done in ordinary cases.

At present, the conventional way of designing fire-exposed steel structures analytically assumes a uniformly distributed temperature along, as well as across, the steel members at each time of thermal exposure. The ECCS Recommendations [1] follow this kind of approach. The heat transfer equations for calculating the steel member temperature then, generally give a uniformly distributed temperature along and across the structure which, at each time of thermal exposure, approximately coincides with the maximum steel temperature, measured in a corresponding test. This is
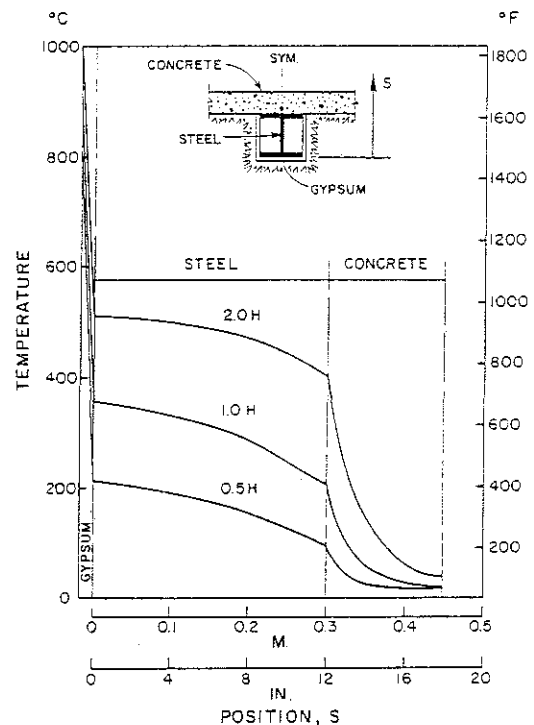

Fig. 4. Calculated temperature distribution along line of symmetry of a steel beam, insulated by a 16 mm gypsum board (density 770 kg m^{-3}) and carrying a 150 mm concrete slab on the top flange, at selected times of thermal exposure according to eqn. (2.1) [9].
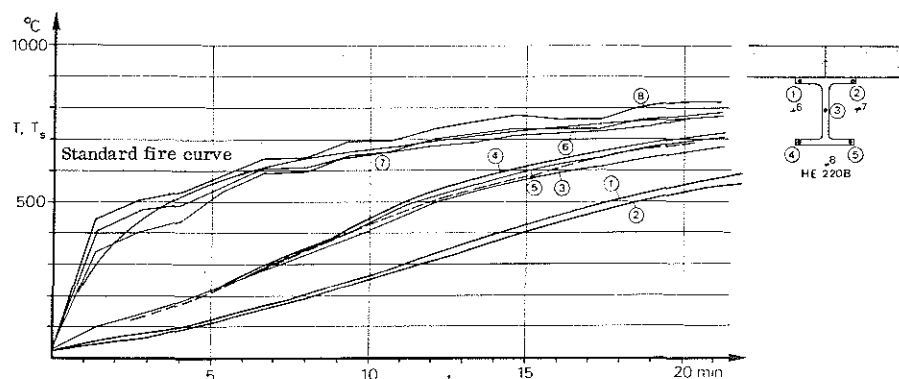
Fig. 5. Calculated, uniformly distributed temperature T_s (— — — —) compared with temperatures measured at different points of cross section of an uninsulated steel girder (————) during a fire resistance test [10].

illustrated by Fig. 5 [10], in which the calculated steel temperature–time curve is compared with the temperature–time curves measured in a fire resistance test at different points of the cross section, of an uninsulated steel beam with a concrete slab on the top flange. The calculated steel temperature is in good agreement with the temperature measured in the lower part of the steel beam section. The measured temperature in the top flange is consistently lower than in the rest of the girder. This is due to the fact that the top flange is exposed to less direct radiation than the bottom flange, and also that there is a continuous conduction of heat away from the top flange of the girder into the cooler concrete slab.

3.5. Restraint of the structural element

Analytical methods of structural fire engineering design enable rotational as well as axial restraints of a structural steel element to be taken into account. Rotational restraints of steel beam elements can then be dealt with in a rather simple way by using a limit state design according to the elementary plastic theory. A consideration of the axial restraint of a steel member from adjacent structural members usually requires a more advanced analysis, and use has to be made of computer programmes.

4. DISCREPANCIES IN FIRE RESISTANCE, DETERMINED BY FIRE RESISTANCE TESTS AND BY CALCULATION ACCORDING TO ECCS RECOMMENDATIONS

The comparative discussions presented in Sections 2 and 3 have shown that consider-

able discrepancies arise when the fire resistance of a steel structure or structural element is determined on the one hand by a standard fire resistance test, and on the other by a calculation according to the approach specified in, for instance, the ECCS Recommendations [1]. Generally, then, the analytical method gives a lower value of fire resistance than the test method. Related to the concept of the critical temperature of a structural steel element, the analytically determined value will be substantially lower than the corresponding value measured in a standard fire resistance test.

The main reasons for the discrepancies can be summarized as follows:

(1) Analytical methods are based on the characteristic values of the mechanical material properties at elevated temperatures, whereas fire resistance tests are performed on specimens whose material properties are random samples;

(2) in the conventional analytical approach, the temperature is assumed to be uniformly distributed along and across the structural steel member, whereas a considerable non-uniformity in the steel temperature distribution can arise in fire resistance tests;

(3) imperfections, which have an important influence on the load-bearing capacity for columns, constitute a random sample for a test specimen in a fire resistance test, whereas in an analytical design the imperfection parameter is chosen — according to code practice — in such a way that the failure load coincides with the 2.3% confidence level.

Evidently, similar discrepancies would exist if test results were compared with analytical results in a structural design for ordinary room

temperature conditions. A structural design, directly based on the test results for room temperature conditions, however, is done at present only in exceptional cases. In a structural fire engineering design, the fire resistance test will also be frequently used for years to come, even if an analytical solution gradually becomes more common in practice. Consequently, there is an urgent need to develop a method of avoiding the described discrepancies and achieving consistency between the analytical and experimental approaches. The following alternatives then can be seen as possible:

(1) To transform the results of fire resistance tests on specimens with random material properties and conditions of non-uniform temperature distribution to values which are related to the characteristic values of the material properties, and to a uniformly distributed temperature along and across the structural steel member. For columns, an additional adjustment has to be made with respect to imperfections;

(2) to base the analytical determination of the fire resistance on mean values or some other representative values — instead of characteristic values — of the structural material properties and the imperfections of the structural member. For steel beam members, a positive correction is allowed for the influence of a steel temperature variation along and across the member. For columns, such a correction is less important;

(3) to base the analytical determination of the fire resistance on a lower load level than the design load value, e.g., the dead load plus some part of the characteristic value of the live load.

A comparison shows the first alternative to be the most consistent one. This alternative, however, has the serious disadvantage of drastically changing a test and classification procedure which has been frequently used on an international level for a very long time. Such a changed procedure for the future, however, would result in lower values of the structural fire resistance than those accepted at present. Moreover, the adjustments of the test results required will be far from simple to perform.

The second alternative is comparatively simple to prepare for practical use, but if it is extended to temperature levels which approach room temperature it provides a direct contradiction to the ordinary structural design for room temperature conditions which is based on modern reliability theories.

The third alternative is, in principle, more consistent with modern reliability theories. However, it introduces different loading, imperfection, and temperature conditions for a design based on tests and a design based on an analytical approach. The load level will vary also with the type of structure and the kind of structural material. Moreover, the long standing rule, that the load during a fire shall be assumed equal to the design load, will probably not be easy to change.

If we seek a solution, which can be applied in practice immediately, the preference must be for the second alternative. For that reason, this alternative will be dealt with further in the next Section.

5. METHOD OF ACHIEVING CONSISTENCY BETWEEN ANALYTICAL AND EXPERIMENTAL APPROACHES

In the previous Section, the discrepancies in fire resistance, determined on the one hand analytically, and on the other by a standard fire resistance test, were discussed summarily with particular application to load-bearing steel structures or structural members. Three choices of method were presented for improving the consistency between the results from the analytical and from the experimental approaches by a correction of either the test results or the results of an analytical solution. A comparison then led to the second choice as being best applied to an immediate practical application.

The alternative can be described as built up of two design steps. In the first step, the load-bearing capacity of the steel structure or structural member, R, is computed for the required time of fire resistance, assuming

(1) mechanical properties of the structural materials at elevated temperatures, which are given by the characteristic values;

(2) imperfections of the structure or structural member, which coincide with the characteristic values;

(3) a steel temperature which is uniformly distributed over the cross section of the structure at each time of thermal exposure;

(4) a steel temperature which is also uniformly distributed along the structure at each time of thermal exposure.

The load-bearing capacity R, calculated under these assumptions, is then multiplied in a second step by a factor of magnification, f, which includes corrections in respect of representative deviations from the assumptions listed for the real structure or structural element. The corrected load-bearing capacity

$$R_c = fR \qquad (5.1)$$

obtained in this way, can be considered as approximately consistent with the corresponding load-bearing capacity, determined in a standard fire resistance test.

The fire engineering design criterion is that

$$R_c > S \qquad (5.2)$$

where S is the design load effect on the structure.

With regard to the listed assumptions, the magnification factor, f, can be written as

$$f = f_m f_i f_{Tc} f_{Ta} \qquad (5.3)$$

where f_m = a correction factor related to the mechanical properties of the structural materials at elevated temperatures, f_i = a correction factor related to the imperfections of the structure, f_{Tc} = a correction factor in respect of non-uniformity in the temperature distribution over the cross section of the structure, and f_{Ta} = a correction factor in respect of non-uniformity in the temperature distribution along the structure.

A set of representative values is given below for the various correction factors. The presentation is summed up by one diagram which gives the resultant correction factor, f, according to eqn. (5.3), and one diagram with corresponding formulae, which approximate the resultant correction factor in a simplified manner. This simplification is quite deliberate. Representative correction factor values, which are progressing towards improved agreement between analytically and experimentally determined fire resistance, cannot be defined with high accuracy in the present state of knowledge. The Section concludes with two supplementary diagrams for an alternative approach to the correction procedure, which is numerically equivalent to the procedure described by eqns. (5.1) - (5.3) but is more precisely adapted to the ECCS Recommendations [1].

5.1. Correction factor f_m

In Section 3.2., the value 1.2 was stated as being representative of the quotient between the mean value $\bar{M}$ and the characteristic value $\bar{M} - 2\sigma_M$ of the yield stress for structural mild steels at ordinary room temperature. The same value can also be considered as being roughly representative of the yields stress at elevated temperatures.

Choosing the mean value of the yield stress as a basis for an analytical determination of the fire resistance of a steel structure can be shown to give a too favourable result as compared with a corresponding result from a standard fire resistance test. Consequently, a value smaller than 1.2 should be chosen for the correction factor f_m. The value $f_m = 1.1$, which approximately corresponds to the quotient $(\bar{M} - \sigma_M)/(\bar{M} - 2\sigma_M)$, then seems to be reasonable. The correction factor applies to all types of steel structures or structural members. In order not to give a direct contradiction to the ordinary structural design for room temperature conditions, which is based on the characteristic value of the material strength, the application of the correction factor f_m should be limited to a range of steel temperatures, T_s — say $T_s \geqslant 300$ °C — which is representative of a fire exposure. With a linear variation of f_m between $T_s = 0$ and 300 °C, this gives for f_m the relationship

$$f_m = 1 + \frac{1}{3\,000} T_s \qquad \text{for } 0 \leqslant T_s < 300 \text{ °C}$$

$$f_m = 1.1 \qquad \text{for } T_s \geqslant 300 \text{ °C.}$$

$$(5.1.1)$$

5.2. Correction factor f_i

The points of view put forward in the previous Section concerning the correction factor f_m are, in principle, mainly applicable to the correction factor f_i which is related to the imperfections of the structure or structural member. Insufficient knowledge, especially concerning the influence of imperfections on structural behaviour at elevated temperatures, does not allow a choice of any other relationship for f_i than the one given for f_m by eqn. (5.1.1), i.e.,

$$f_i = 1 + \frac{1}{3\,000} T_s \qquad \text{for } 0 \leqslant T_s < 300 \text{ °C}$$

$$f_i = 1.1 \qquad \text{for } T_s \geqslant 300 \text{ °C.}$$

$$(5.2.1)$$

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The correction factor f_i applies only to such types of steel structures or structural members for which the initial imperfections decisively influence the load-bearing capacity, primarily columns.

5.3 Correction factor f_{T_c}

An analytical determination of the fire resistance of a steel structure according to, for example, the ECCS Recommendations [1] assumes a steel temperature T_s, which is uniformly distributed over the cross section and the length of the structure at each time of thermal exposure. The heat transfer equations presented in the Recommendations for a calculation of this temperature give a value which approximately coincides with the maximum steel temperature measured in a corresponding fire resistance test (see Fig. 5 [10]).

For steel columns, which are exposed to fire on all sides, the assumption of a uniformly distributed temperature over the cross section describes the real situation in an acceptable way.

For steel beams thermally exposed in a standard fire resistance test, there is a considerable temperature variation over the height of the cross section. For steel beams protected by either a suspended ceiling or by an insulation surrounding the steel profile, accurate calculations indicate a temperature difference between the bottom flange and the top flange of the order of 100 - 150 °C at standard fire exposure conditions (see Fig. 4 [9]). This is also confirmed by standard fire resistance tests, *cf.*, for instance, Fig. 1 [5]. For unprotected steel beams, the corresponding temperature difference is somewhat smaller.

The correction factor f_{T_c} gives, for a steel beam, the quotient between the load-bearing capacity at a steel temperature which is lower in the top flange than in the bottom flange, and the load-bearing capacity at a uniformly distributed temperature over the cross section. For both load-bearing capacities, then, the temperature is assumed to be equal in the bottom flange.

A limit state design according to the elementary plastic theory constitutes one way of calculating the correction factor f_{T_c}. For an I-shaped steel-beam-cross-section with a temperature which is 100 °C smaller in the top flange than in the bottom flange, such a deter-

mination gives the dashed curve with circles in Fig. 6. The curve has been derived from analytical results presented in ref. 11. For a bottom flange temperature of less than about 300 °C, it seems justifiable to make $f_{T_c} \cong 1$.

Alternatively, the correction factor f_{T_c} can be determined by computing the deflection curve of the thermally exposed steel beam and defining the ultimate load-bearing capacity by a limit deflection criterion. The full-line curves in Fig. 6 are derived in this way by directly applying the theoretical results presented in ref. 6. With the ultimate load-bearing capacity defined with regard to a limit deflection of the steel beam, the correction factor f_{T_c} will be dependent on the type of structural system. The two curves shown in Fig. 6 refer to a simply supported and a built in steel beam, respectively.

5.4. Correction factor f_{T_a}

In standard fire resistance tests of beams, the temperature at the supports can be considerably less than the simultaneous temperature in the centre of the span. This situation also applies to the conditions of a real fire exposure. The temperature difference between the centre of the span and the support regions is usually of the order of 100 - 200 °C but may sometimes be substantially larger (see Fig. 1).

For isostatic steel beams, the influence of a temperature variation along the beam on the load-bearing capacity is of minor importance
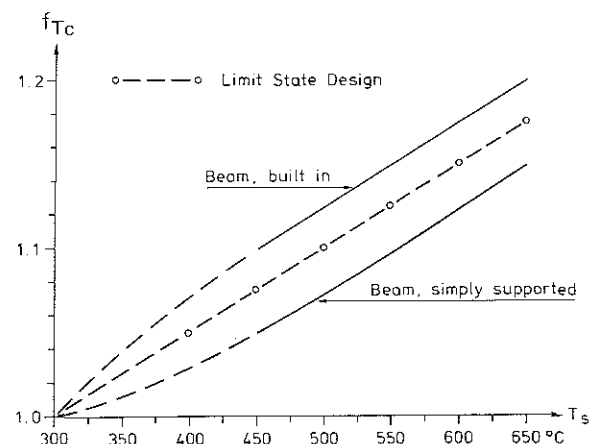

Fig. 6. Correction factor, f_{T_c}, for steel beams, determined either according to elementary plastic theory (dashed curve with circles) or by computing the deflection of the beam and defining the ultimate state by a limit deflection (full-line curves). The curves are derived from analytical results presented in refs. 6 and 11.

and can, as a rule, be neglected. For hyperstatic steel beams, the influence is directly decisive and consequently must be considered. This can be done using the correction factor f_{Ta}.

The correction factor f_{Ta} gives, for a steel beam, the quotient between the load-bearing capacity at a steel temperature which is lower at the supports than in the centre of the span, and the load-bearing capacity at a uniformly distributed temperature along the structure. For both load-bearing capacities, the steel temperature is assumed to be equal in the centre of the span.

For a steel beam, which is built in at the supports and for which the steel temperature is 100 °C smaller at the supports than in the centre of the span, a limit state design according to the elementary plastic theory gives the dashed curve with circles in Fig. 7 for the correction factor f_{Ta}. The curve has been computed on the assumption of a temperature dependent effective yield stress σ_{yT} according to Fig. 3 [1]. The full-line curve shows the corresponding relationship between the correction factor f_{Ta} and the steel temperature at the centre of the span T_s, determined by computing the deflection curve of the hyperstatic beam and defining the ultimate state by a limit deflection [6]. For a steel beam temperature of less than about 300 °C, it seems justifiable to make $f_{Ta} \cong 1$.

The f_{Ta} values according to Fig. 7 are for a hyperstatic structure with two redundancies. For a hyperstatic structure with only one redundancy, e.g., a continuous beam on three supports, the correction factor can be chosen as approximately $1/2(1 + f_{Ta})$.

5.5. Resultant correction factor, f

With known values of the correction factors f_m, f_i, f_{Tc} and f_{Ta}, the resultant correction factor, f, is obtained from eqn. (5.3). Correction factor f_m, as given by eqn. (5.1.1), applies to all types of steel structures and correction factor f_i, as given by eqn. (5.2.1), applies to steel columns. For beams $f_i \cong 1$. Correction factor f_{Tc}, as given by Fig. 6, applies to isostatic and hyperstatic steel beams. For steel columns $f_{Tc} \cong 1$. Finally, correction factor f_{Ta}, as given by Fig. 7, applies to hyperstatic steel beams. For isostatic beams and columns $f_{Ta} \cong 1$.

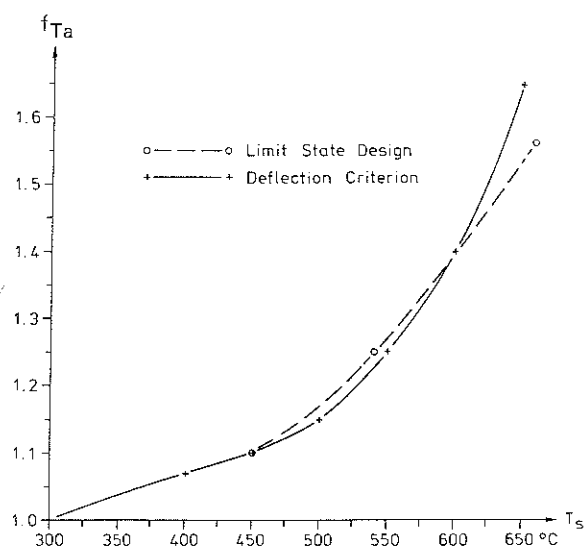Fig. 7. Correction factor, f_{Ta}, for hyperstatic steel beams, determined either according to elementary plastic theory (dashed curve with circles) or by computing the deflection of the beam and defining the ultimate state by a limit deflection (full-line curve). The second curve is derived from analytical results presented in ref. 6. The values apply to hyperstatic beams with two redundancies. For hyperstatic beams with only one redundancy, the correction factor can be chosen as $1/2(1 + f_{Ta})$.
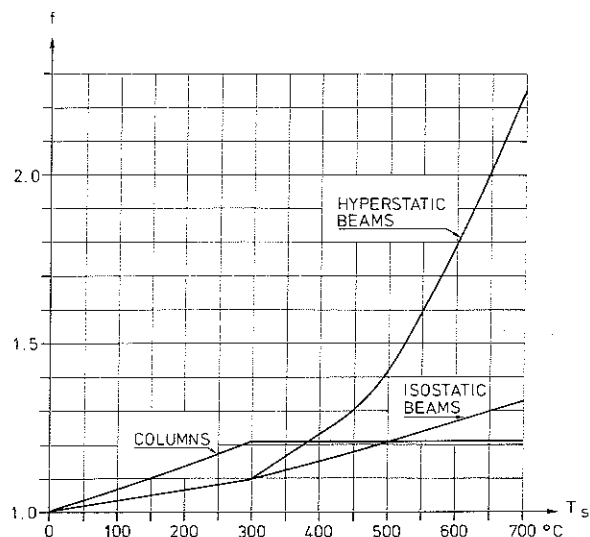

Fig. 8. Resultant correction factor, f, for load-bearing capacity, R, as a function of uniformly distributed calculated steel temperature, T_s, for columns, isostatic beams, and hyperstatic beams with two redundancies. For hyperstatic beams with only one redundancy, f can be chosen as approximately the average of the values for isostatic beams and hyperstatic beams with two redundancies.

Fig. 9. Simplified relation between resultant correction factor, f, and uniformly distributed steel temperature, T_s, for columns, isostatic beams, and hyperstatic beams with two redundancies. For hyperstatic beams with only one redundancy, f can be chosen as approximately the average of the values for isostatic beams and hyperstatic beams with two redundancies.
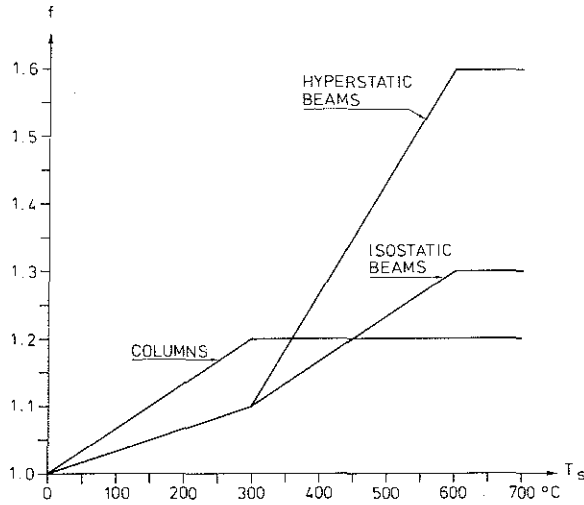
If the values of the correction factors f_m, f_i, f_{Tc} and f_{Ta} are taken according to eqns. (5.1.1) and (5.2.1), and Figs. 6 and 7 (dashed curves), respectively, the curves in Fig. 8 are obtained for the resultant correction factor, f, as a function of the calculated uniformly distributed steel temperature, T_s, for columns, isostatic beams and hyperstatic beams.

Roughly approximated, the f-curves according to Fig. 8 can be replaced by the straight line relationships shown in Fig. 9 and described by the formulae:

For columns

$$f = 1 + \frac{1}{1\,500}\,T_s \qquad \text{for } 0 \leqslant T_s < 300\ ^{\circ}\text{C}$$

$$f = 1.2 \qquad\qquad \text{for } T_s \geqslant 300\ ^{\circ}\text{C}$$

$$(5.5.1)$$

For isostatic beams

$$f = 1 + \frac{1}{3\,000}\,T_s \qquad \text{for } 0 \leqslant T_s < 300\ ^{\circ}\text{C}$$

$$f = 1.1 + \frac{1}{1\,500}\,(T_s - 300)$$

$$\text{for } 300 \leqslant T_s < 600\ ^{\circ}\text{C}$$

$$f = 1.3 \qquad\qquad \text{for } T_s \geqslant 600\ ^{\circ}\text{C}$$

$$(5.5.2)$$

For hyperstatic beams

$$f = 1 + \frac{1}{3\,000}\,T_s \qquad \text{for } 0 \leqslant T_s < 300\ ^{\circ}\text{C}$$

$$f = 1.1 + \frac{1}{600}\,(T_s - 300)$$

$$\text{for } 300 \leqslant T_s < 600\ ^{\circ}\text{C}$$

$$f = 1.6 \qquad\qquad \text{for } T_s \geqslant 600\ ^{\circ}\text{C}$$

$$(5.5.3)$$

Figures 8 and 9 and eqns. (5.5.1) - (5.5.3) summarize the design basis required for the correction of the load-bearing capacity, R, of a structural steel member, computed for a prescribed time of fire resistance on the assumption of characteristic values of mechanical properties of the structural material, characteristic values of the imperfections of the structural member, and a uniformly distributed steel temperature over the cross section and the length of the structural member. The applicable value of the resultant correction factor, f, then, by way of eqn. (5.1), transforms the load-bearing capacity R to the modified quantity R_c, which can be considered as being approximately consistent with the corresponding load-bearing capacity derived from the results of a standard fire resistance test.
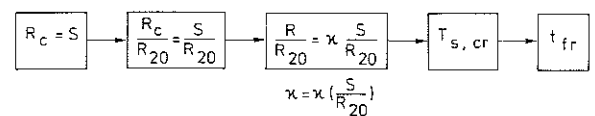
5.6. A slightly modified approach

The method of correction, presented in subsections 5.1 - 5.5, has been directly adapted to a design procedure according to Fig. 10 and eqns. (5.1) - (5.3).
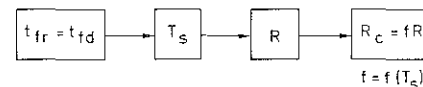
DESIGN CRITERION: $R_c > S$

Fig. 10. Procedure for an analytical fire engineering design, based on the concept of the fire resistance of a load-bearing structure.

DESIGN CRITERION: $t_{fr} > t_{fd}$

Fig. 11. Alternative procedure for an analytical fire engineering design, based on the concept of the fire resistance of a load-bearing structure.

The procedure starts from the required time of fire duration, t_{fd}, for which the structure has to fulfil its load-bearing function. For a thermal exposure according to eqn. (2.1), with an endurance $t_{fr} = t_{fd}$, the ECCS Recommendations [1] or any other equivalent design basis give a uniformly distributed steel temperature, T_s, of the structure. This temperature is transferred analytically to a load-bearing capacity, R, based on the characteristic values of the mechanical properties of the material and on the characteristic values of the imperfections of the structure. Finally, the load-bearing capacity R is corrected by the use of Figs. 8 or 9 to the value R_c to obtain a modified load-bearing capacity which is more consistent with the connected load-bearing capacity obtained in a standard fire resistance test. The resultant correction factor, f, is a function of the uniformly distributed steel temperature T_s. The design criterion has the form $R_c > S$, where S is the design load effect on the structure.

Alternatively, the design procedure can be carried out in conformity with Fig. 11 which describes a sequence more directly related to the ECCS Recommendations [1].

The procedure starts with the corrected load-bearing capacity of the thermally exposed structure, R_c, made equal to the design load effect on the structure, S. This defines a quotient, $R_c/R_{20} = S/R_{20}$ where R_{20} is the load-bearing capacity of the structure at room temperature, calculated on the basis of the characteristic values of the mechanical properties of the material and of the characteristic values of the imperfections of the structure. The next step of the design procedure comprises a determination of the quotient

$$\frac{R}{R_{20}} = \kappa \frac{S}{R_{20}} \tag{5.6.1}$$

where $R = R_c/f$ (see eqn. (5.1) or Fig. 10). The multiplier, $\kappa = 1/f$, can then be specified as a function of the uniformly distributed steel temperature T_S, the quotient R/R_{20} or the quotient S/R_{20}. From Fig. 11, it is evident that a trial and error step is avoided in the design procedure if κ is given as a function of S/R_{20}. At a known value of

$$\frac{R}{R_{20}} = \frac{\sigma_{yT}}{\sigma_{y,20}} \tag{5.6.2}$$

the diagram in Fig. 3 determines the corresponding, uniformly distributed, critical steel temperature $T_{s,cr}$, which can easily be transferred to the fire resistance of the structure t_{fr}. As a consequence of applying the corrected load-bearing capacity of the structure R_c as basic input data for the design procedure, the calculated fire resistance t_{fr} can be considered as approximately consistent with the connected fire resistance derived from a standard fire resistance test.

The design criterion of the alternative design procedure has the form $t_{fr} > t_{fd}$, where t_{fd} is the time of fire duration required in the building codes for the structural application in question.

Figure 12 presents the relationship between the multiplier κ and the quotient $S/R_{20} = R_c/R_{20}$ for columns, isostatic beams and hyperstatic beams of steel. The κ-curves directly correspond to the f-curves in Fig. 8. The connection is regulated by the equation $\kappa = 1/f$ and by Fig. 3, combined with eqn. (5.6.2).

The κ values according to Fig. 12 imply a hyperstatic structure with two redundancies. For a hyperstatic structure with only one redundancy, the multiplier κ roughly can be
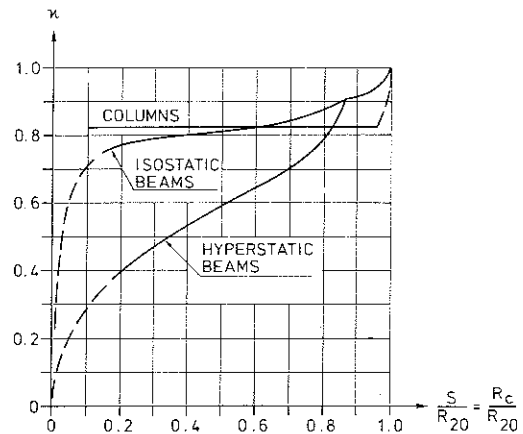

Fig. 12. Multiplier κ as a function of quotient S/R_{20} for columns, isostatic beams, and hyperstatic beams of steel. S is the design load effect and R_{20} is the load-bearing capacity of the structure at ordinary room temperature, calculated on the basis of the characteristic value of the mechanical strength of the material and of the characteristic value of imperfections of the structure. The values for hyperstatic beams require two redundancies. For hyperstatic beams with only one redundancy, κ can be chosen approximately as the average of the values for isostatic beams and hyperstatic beams with two redundancies.

Fig. 13. Simplified relationships between multiplier, κ, and quotient S/R_{20} for columns, isostatic beams, and hyperstatic beams with two redundancies. For hyperstatic beams with only one redundancy, κ can be chosen approximately as the average of the values for isostatic beams and hyperstatic beams with two redundancies.
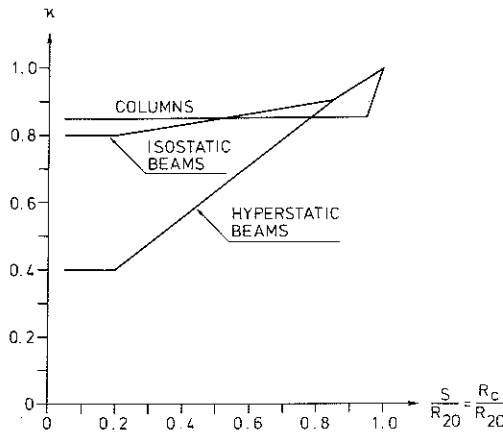
chosen as the average of the values for isostatic beams and hyperstatic structures with two redundancies.

The straight line curves in Fig. 13 are rough approximations of the more accurate curves in Fig. 12. The straight line curves are described by the following formulae:

For columns

$$\kappa = 0.85 \qquad \text{for } 0 \leqslant \frac{S}{R_{20}} < 0.95$$

$$\kappa = -2 + 3\,\frac{S}{R_{20}} \qquad \text{for } 0.95 \leqslant \frac{S}{R_{20}} \leqslant 1$$

$$(5.6.3)$$

For isostatic beams

$$\kappa = 0.80 \qquad \text{for } 0 \leqslant \frac{S}{R_{20}} < 0.2$$

$$\kappa = 0.77 + 0.15\,\frac{S}{R_{20}} \qquad \text{for } 0.2 \leqslant \frac{S}{R_{20}} < 0.85$$

$$\kappa = 0.33 + 0.67\,\frac{S}{R_{20}} \qquad \text{for } 0.85 \leqslant \frac{S}{R_{20}} \leqslant 1$$

$$(5.6.4)$$

For hyperstatic beams

$$\kappa = 0.40 \qquad \text{for } 0 \leqslant \frac{S}{R_{20}} < 0.2$$

$$\kappa = 0.25 + 0.77\,\frac{S}{R_{20}} \qquad \text{for } 0.2 \leqslant \frac{S}{R_{20}} < 0.85$$

$$\kappa = 0.33 + 0.67\,\frac{S}{R_{20}} \qquad \text{for } 0.85 \leqslant \frac{S}{R_{20}} \leqslant 1$$

$$(5.6.5)$$

LIST OF SYMBOLS

$\bar{M}$	mean value
R	load-bearing capacity of structure at elevated temperatures
R_{c}	corrected value of R
R_{20}	load-bearing capacity of structure at ordinary room temperature
S	design load effect on structure
T	temperature
T_{o}	furnace temperature at time $t = 0$
T_{s}	steel temperature
$T_{\mathrm{s,cr}}$	critical steel temperature
f	factor of magnification $(=R_{\mathrm{c}}/R)$
f_{i}	correction factor, related to imperfections of structure
f_{m}	correction factor, related to mechanical properties of structural material at elevated temperatures
f_{Ta}	correction factor with respect to non-uniformity in temperature distribution along structure
f_{Tc}	correction factor with respect to non-uniformity in temperature distribution across structure
s	position co-ordinate
t	time
t_{fd}	fire duration
t_{fr}	fire resistance, related to thermal exposure according to standard fire resistance tests
κ	$1/f$
σ_{M}	standard deviation
σ_{yT}	effective yield stress at temperature T
$\sigma_{\mathrm{y,20}}$	yield stress at ordinary room temperature

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