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DESIGN PHILOSOPHY FOR FIRE EXPOSED CONCRETE STRUCTURES

CONTENTS

Thaddeus Hyatt

PREFACE

One of the most versatile intellects in the history
of structural fire research was the American lawyer
Thaddeus Hyatt (1816-1901).

Besides fighting against negro slavery, writing about
the possibility of making a propeller-driven aircraft
and much more he found that unprotected steel struc-
tures were not so fire proof as they were believed
to be at the time.

During the 1870'es he tested several load-bearing
reinforced concrete constructions for fire exposure,
and he developed many fire proof details.

He even built a house at Farringdon Road in London
using the principles found and set it on fire as a
public demonstration. (Demolished in 1913).

By these activities he must be counted as a real
pioneer of fire safety design of concrete construc-
tions, which is the subject of this report.

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SYMBOLS

A	area
h	height of an opening
q	fire load
T	temperature
t	time

Indices

t	total

ACKNOWLEDGEMENTS

SUMMARY

From an analysis of the purposes of doing a fire
safety design for load-bearing concrete structures
conclusions are made concerning the requirements
which the design procedures should fulfil.

When the safety level is found from the basic requi-
rements, fire can be treated as a load case, which
should fit into the total system of load cases used
for the structural design.

It is concluded that the fire exposure used should
be related to the destructive potential of the real
fire, and the design procedures should correspond
to the procedures used in other load cases, if the
fire exposure is varied and approaches nil.

Further it is found that plastic design procedures
are especially suitable to fire exposed concrete
structures, and if the structural plasticity allows
such methods to be used in any other load case, they
will also be allowable in the case of fire.

The relationship between calculation and testing is
discussed, and in case these procedures should be
complementary to one another, they should be based
on the same conditions, which first of all means
the same loads, the same dimensions and fire courses
of the same destructive potential.

This indicates a necessary development for the testing
institutions towards the application of flexible
light weight furnaces for testing of structural ele-
ments of full size exposed to variable fully de-
veloped fire courses.

The origin of the standard fire curve is briefly
mentioned and the necessity of different heating
curves is discussed, and the serious consequences
of not applying a cooling phase are emphasized.

It is proposed to apply a number of standardized
fully developed fire curves, and the idea is discussed
of taking into account the effect of a fire brigades
efforts or the effect of an automatic extinguishing
system defining the design requirements.

THE BASIC REQUIREMENTS

Since the times of Thaddeus Hyatt fire safety design of load-bearing constructions in practice has been based mainly on results of full-scale testings.

In order to make the test results of different test institutions and different structures mutually comparable, the test procedures were standardized at the beginning of the century.

A system of classification was developed using the concept of "Time of fire resistance" meaning the maximum time of standard fire exposure, which the test specimen is able to withstand.

According to some national standards the structure may be unloaded immediately after the fire exposure, and according to others the load should be carried for 24 hours after the fire exposure.

This difference has only been paid very little attention. So little that for example the 24 hour requirement in the Danish standard DS 1051, the edition of 1966 has been deleted by the 1979 edition.

However, as it will be shown, the 24 hour rule is essential to the correlation between a standard fire test and a real fire course.

Because of its easy application and simple concepts, the classification system has become rigid in use, and the definition of requirements as well as their fulfilment has become matters of administration, which has a decreasingly important relationship to physical reality.

Therefore, it is due time for considering the fire safety design as a whole again.

The use of structural fire tests and calculations does make sense only if it improves the fire safety design to fulfil at least the most fundamental requirements, which are the generally accepted reasons for making a fire safety design:

TO SAVE LIFE AND PROPERTY

The benefits of an investment in a fire safety design are closely related to the fulfilment of these requirements stated by our politicians and by many clients making the investment.

This means that it is not sufficient only to save lives, and it is not sufficient to ensure that the structure will be able to withstand a certain time of fire exposure in order to secure the escape of people being in the building, when fire breaks out.

By doing so, the lives may not be saved for those, who enter the building to fight the fire or to save properties or later on to restore the building.

By doing so, neither the building itself nor the values it contains may be saved, and for a factory or a trading company the sudden lack of potential for production or delivery often means a loss of its market share, i.e. loss of property.

It is therefore a reasonable requirement that the structure should be able to withstand a full fire course without having an insufficient load-bearing capacity at any time of the fire course.

The load-bearing capacity, which is considered to be sufficient, must be related to the safety level needed.

Detailed requirements could be specified by the building authorities, the insurance companies, the clients or the consultants as specific requirements for each single part of the construction or as a part of a total fire safety design for the building.

The fire safety design could be made as an economical optimization relating costs and benefits, or the safety level could be determined from other considerations.

No matter how the safety level is found, fire can be considered as a load case, and the fire course used for the purpose of design should, as close as possible, have the same destructive potential as the real fire that the structure may be exposed to.

Furthermore, the procedure of structural design for the fire load case should fit as close as possible to the procedures used for the other load cases, such as ultimate load and wind load etc.

This means that the design for fire exposure should be easy to incorporate in the design of the building, and that the accuracy of the design methods for the load case "Fire" should harmonize with the accuracy used for the other load cases.

For example, it would make no sense, if the static model used for calculation of the load-bearing capacity of a structure in case of fire is more complicated and more precise than the models used for the other load cases.

ULTIMATE LIMIT STATE AND PLASTIC DESIGN

Especially for concrete constructions the ultimate
limit state analysis and plastic considerations has
become very popular for practical design, because the
plastic models are often relatively easy to use, and
the load bearing capacity of a structure can be pre-
dicted with a sufficient accuracy by means of them.

At high temperatures the concrete becomes soft, the
ultimate strain increases considerably, and large
deformations can take place.

The stress-strain relationship will be discussed in
more detail later, but here, the fact that the heated
concrete becomes more plastic is sufficient for making
the conclusion that *if it is allowable to determine
the load-bearing capacity of a structure without fire
exposure by means of an ultimate limit state analysis,
this method of analysis will be at least just as
allowable for the fire exposed structure.*

This conclusion and the advantage of fitting the
design for fire exposure into the design for the
other load cases leads to the statement that *if
plastic design is used to find the load-bearing
capacity in any other load case, it should also be
used in the case of fire.*

In addition the use of plastic design methods often
facilitates the calculational treatment of struc-
tures that causes special problems, when exposed to
fire, such as hyperstatic structures, frames and
shell constructions.

Consequently, there is a need of methods for ultimate limit state design of fire exposed concrete structures based on the same principles as the methods used for the load cases without fire exposure.

To ensure homogeneousness of the design procedures, the calculated load-bearing capacity of a fire exposed structure should approach the one of a similar structure without fire exposure, when the fire exposure approaches nil, and the safety level is presumed constant.

This is obtained if the methods of calculation are identical for load cases without fire exposure and for the case of fire, where the fire exposure is substituted by the constant temperature 20°C.

By increasing the fire exposure, only the material properties changes, and new values of the properties should be applied in the calculation of the load-bearing capacity.

The compressive strength, the tensile strength, the modulus of elasticity as well as the thermal conductivity, the heat capacity and the density changes for concrete and steel.

The changes in density will often cause thermal stresses, which could be applied as extra loads on the cross section considered.

Because of the large temperature differences the material properties varies considerably over a given cross section; thus there is a need of simple concepts, by means of which these variations can be handled and used for extended versions of the common pro-

cedures for calculation the load-bearing capacity
of the cross section in the ultimate limit state.

The extensions should give no changes of the common
procedures in case the fire exposure is reduced to
nil.

In this way, the common procedures of calculation
become marginal cases of the procedures for calcu-
lating the load-bearing capacities of fire exposed
structures.

By means of the extended procedures it should be
possible to calculate the load-bearing capacities at
any time of a full developed fire course.

By doing so, and observing the temporal development,
it is usually seen that the load-bearing capacity
becomes a minimum at two or more stages of the fire.

For example for a cross section subjected to bending
this happens when the reinforcement strength is a
minimum and when the concrete strength is a minimum.

Being able to predict the times, when this happens,
the minimum load-bearing capacity of the cross section
during the whole fire course is found as the smaller
capacity occurring at the times of local minima.

Finally, if the methods of calculation allow estima-
tion of the load-bearing capacity at any time of a
fire course, they can also be used for estimating the
minimum load-bearing capacity during a standard fire
exposure without a cooling phase.

In this way the methods can be used to predict the
results of standard fire tests or to verify by cal-
culation the fire resistances still required by many

national authorities.

Further, this gives the possibility of using the large
amount of data from standard fire tests to verify the
calculation methods applied on these abrupt fire
courses.

In short, ultimate limit state design for fire ex-
posure should represent simple extensions of the de-
sign methods used in other load cases, and result
in the minimum load-bearing capacity of the structure
considered, when exposed to a full fire course, as
well as at any specific time of a natural or standar-
dized fire course.

CALCULATION AND TESTING

The purpose of making a structural fire test is to estimate the load-bearing capacity of the fire exposed structure, which is the same as the purpose of making a similar calculation, but in addition any unexpected phenomena arising can be observed, which may influence the function, the maintenance or the reconstruction of the structure.

Also, since the calculational models often are conservative, a fire test may often lead to a better fire resistance than possible by calculation.

Therefore, although fire testings always will be costly and time consuming, they are necessary for research and of great value for design purposes.

Testings can especially be valuable for design of structural elements, which are intended to be produced in such numbers, by which the costs of the testings are small compared to the benefits of the improved designs.

On the other hand, calculational procedures may give good indications on how to design a new structural element that must be able to withstand a certain fire test, and in this way calculations can reduce the number of fire tests necessary and thereby the total design costs.

These relationships between fire tests and calculations are precisely the same as the relations between structural tests and the corresponding calculations used for any other load case.

Therefore, structural fire tests as well as the

structural tests used as supplements to calculational
design procedures should be based on as close as
possible the same conditions, and involve the same
loads, as used for the calculations.

First of all this means that the size and the loading
of a structural element should be correct when used
for a fire test.

It can easily be shown (Hertz [7]) that scale model-
ing is impossible for fire tests of load-bearing
concrete structures, because the simultaneous fulfil-
ment of the necessary laws of modeling requires that
the scale can only be 1:1, i.e. full scale is required.

Since concrete structures are often very large, the
fire test equipment and especially the furnaces should
also be large, and this leads to severe economical
problems for the testing laboratories.

This means that frequently a calculational design will
be the only economical possibility for many large
concrete structures.

At present many national standards prescribe the
sizes of the test specimens for standard fire tests
in accordance with limited sizes of the test furnaces
at the testing institutions.

For example this means that only about 5 meters of
a 25 meter span beam will be cut out, loaded and
tested.

It is obvious that such an amputated beam can never
be loaded by the correct maximum moment load and
shear load at the same time, and observations con-
cerning deflections or instability cannot be re-

lated to the real beam.

The value of the fire test will be very limited for describing the structural behaviour of the real beam, and the test results cannot be compared to the results of a calculation of the real beam.

Of course a calculation could also be made for the short test beam, and in case the national authorities accept test results of the cut-off specimens, they consequently also have to accept calculations related to structural elements of wrong sizes, which would indeed be a bizarre development for the fire safety design.

A much more reasonable development would be to encourage the testing institutions to meet the requirements of full scale and full size test for example by construction of new lightweight flexible furnaces which can be built from modular components around the structural elements and be provided with an automatic heat controlling system.

Lightweight furnaces can be made at relatively low costs using combinations of insulating materials of different qualities, so that the most expensive materials are used only at the inside layers, where the temperatures are at a maximum, and cheaper materials are used where lower temperatures occur.

In addition lightweight furnaces do require less energy to be heated than heavy ones, and they can be cooled down more easily in accordance with a prescribed cooling curve.

If fire tests are to be used for fire safety design
of concrete structures as an alternative to calcula-
tion, the structural elements should be of full scale
and full size and loaded, supported and insulated
correctly, and the fire exposure used should be re-
levant to and of the same destructive potential as
the one used for the corresponding calculations.

Combined procedures could also be possible by fulfil-
ment of the relevant conditions, for example by
measuring the temperature distributions on a correctly
exposed and insulated specimen and calculating the
load carrying capacity, or by using calculation for
interpolation or extrapolation from a series of
fire testings varying a few parameters of importance.

During the history of fire safety design it has always
been a problem to establish relevant heating conditions
that would answer the exposure from a real fire.

It would therefore be natural in this context to pay
some attention to the philosophical aspects concern-
ing the selection of heating curves for testings
and calculations.

THE FIRE EXPOSURE

In the early days the fire exposure used for test purposes was most random.

It was produced by burning wood or coal and mostly the temperature level was measured for the gas and seldomly for the exposed construction.

It was difficult to make a relation between the test results and the behaviour of the construction exposed by a real fire because of the lack of knowledge about the real fire and about the response of the construction to different fires.

It even involved great difficulties to establish the same heating course from one burn to another, and the testing had most of all the aim of demonstrating that the construction was able to endure a "considerable fire exposure", which perhaps became most clear in those testings, where a complete building was put on fire such as Hyatt's demonstration at Farrington Road.

However, furnaces were built and local standards were developed on how to heat them, but the need of a more common basis was increasing.

In 1917 the American Society of Testing of Materials issued the standard C19 (later E119) prescribing the time-temperature curve known today as the "standard fire" curve.

The curve was the result of several discussions about which temperature developments could be expected in real fires, and it was close to the test curve used by the Underwriters Laboratories (Babrauskas and Williamson [1]).

This curve has been widely used since then and with some modifications adopted in the standards of many countries (DS [3]) and has become an international standard (ISO [4]).

But many actual fires have turned out to show more rapid temperature rises than the standard fire, and it has been suggested applying a number of standard curves for different categories of fire risk (Malhotra [5]).

Especially for the design of off-shore rigs, the need of a rapid fire curve simulating a hydrocarbon fire has become evident, and the so called "Mobil" curve is often used (Warren and Corona [6]).

While many considerations have been made on the heating phase, too little attention has been paid to the cooling phase.

Heavy constructions made of concrete and other materials are heating slowly, and their central parts will often reach their maximum temperatures several hours after the time, at which the temperature of the fire compartment was at a maximum.

Therefore, the minimum load bearing capacity of a concrete construction will always be later, - often much later than the maximum of the compartment temperature, and the size of this time gap is very closely related to the geometry of the construction.

To put it in another way: small cross sections will be closer to their minimum load-bearing capacity than large ones, when the temperature of the fire is at a maximum.

This is the reason why a test or a calculation based
on the requirement of a certain number of minutes fire
resistance of a standard fire without cooling can-
not be used to predict the structural behaviour during
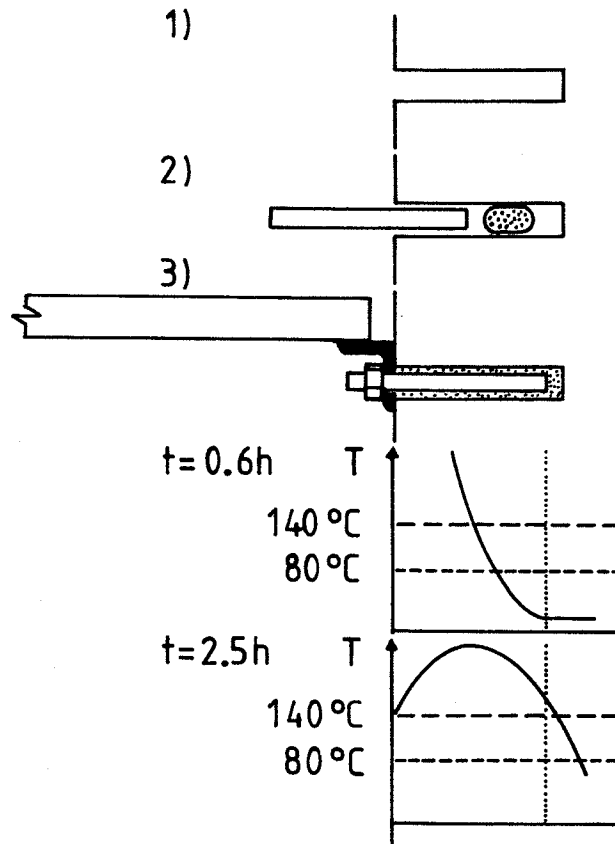a fully developed fire.

Since it has been officially proposed to relate the
destructive potential of a fully developed fire course
to a so called "equivalent time" of a standard fire
exposure without cooling, this concept is discussed
in particular in Hertz [8].

The conclusion is that the concept is of no value
to heavy structures such as concrete structures and
steel structures cast into concrete or protected by
brickwork.

Requirements defined by minutes of fire resistance
at a standardized fire without cooling will lead
to severe differences in resistance to a fully de-
veloped fire, and they will in general be more re-
strictive to small cross sections than to large ones.

In a country, where the requirements of the building
authorities are defined in this way, a design for
fire resistance to meet these requirements will lead
to larger cross sections than necessary in the case
that fire exposure was fully developed, and the larger
cross sections will be more likely to collapse than
the smaller ones.

Consequently the fire losses in such a country will
be larger than if the requirements were formulated
in terms of capacity at fully developed fires, and
the same amount of money was spent on fire protection.

1)

2)

3)

t = 0.6h   T

140 °C

80 °C

t = 2.5h   T

140 °C

80 °C

Principle of an 100 mm adhesive anchor and temperature distributions ($q=200$ MJ/m^2, $A\sqrt{h}/A_t = 0.04$ m$^{\frac{1}{2}}$, quartz).

Furthermore, some serious structural defects may remain undiscovered, in case a fire safety design is made only for a fire exposure without cooling.

One example is the so called adhesive anchor, which has become a commonly used method of connecting structural elements.

A hole is drilled into a concrete surface, and an ampulla with the adhesive is placed in the hole followed by a steel anchor.

A beat on the end of the anchor breaks the ampulla, and the anchor is glued to the inner surface of the hole.

According to investigations made by the producing companies, the bond strength drops to 60 pct. of its initial value, when it is heated to 80^{0}C, and at

$140\,^{\circ}C$ nothing is left of the bond strength.

It is therefore required, that the anchor is made
of such a length that a sufficient part of the anchor
reaches a depth from the surface, at which the tempe-
rature will be less than $80\,^{\circ}C$.

The necessary depth is then found by measuring the
temperatures through a concrete slab at different
times of a standard fire exposure from one side.

In this way anchors are designed to meet requirements
of 30, 60 and 90 minutes of standard fire exposure
without cooling.

However, if the constructions are considered exposed
to real fires, almost none of these anchors will be
less than $140\,^{\circ}C$ in their full depth, because the
heat wave penetrates the concrete long time after
the maximum fire temperature occurs, and most con-
crete sections reach temperatures of more than $140\,^{\circ}C$
at their centerlines even at moderate fire exposures.

Taking into account that the adhesive anchors are of-
ten used in practice at places, where they are im-
portant to the overall stability of the structure,
such as fastenings for steel angles, which serve as
supports for beams and entire deck-constructions, it
becomes quite clear, that a total collapse without
warning will be an obvious possibility at almost
any full developed fire in such places.

Of course this is not an acceptable situation, but
the fire safety design based on a fire exposure
without a cooling phase does not indicate anything
suspicious about the constructions performance.

It has often been suggested to take the effect of fire
brigade efforts or automatic sprinkler systems into
account by using a less severe fire exposure than
the fully developed fire related to the actual fire
load.

Whether this procedure is allowable, or not, seems
to be a political decision, because the risk by doing
so can hardly be quantified.

Strikes in the fire brigades and lack of maintenance
of the sprinkler systems caused by alternations in
the use of the buildings are so random events, that
they cannot be covered by reliable statistics.

Furthermore, in case of catastrophes or actions of
war the reduced fire resistance of a large number
of buildings having all been designed assuming the
presence of fire reducing means, may be crucial for
the attempts to limit the total fire damages in a
city.

If a building is designed for the fully developed
fire course related to its contents of combustible
materials, the task of a fire brigade or any automatic
fire fighting equipment is limited to save lives
and property in the building, and the action can be
more efficient than it would be in case it should
also secure the stability of the structure.

If it is decided, that a construction shall be de-
signed for a reduced fire course due to the effect
of fire brigade efforts or sprinklers, the fire course
still comprises a cooling phase, and special damages
and thermal stresses caused by extinguishing water
should be considered.

In order to reduce the number of calculations and
particularly the number of testings necessary to descri·
be all aspects of the fire resistance of a structure,
various methods have been suggested, by which the de-
structive potential of one fully developed fire can
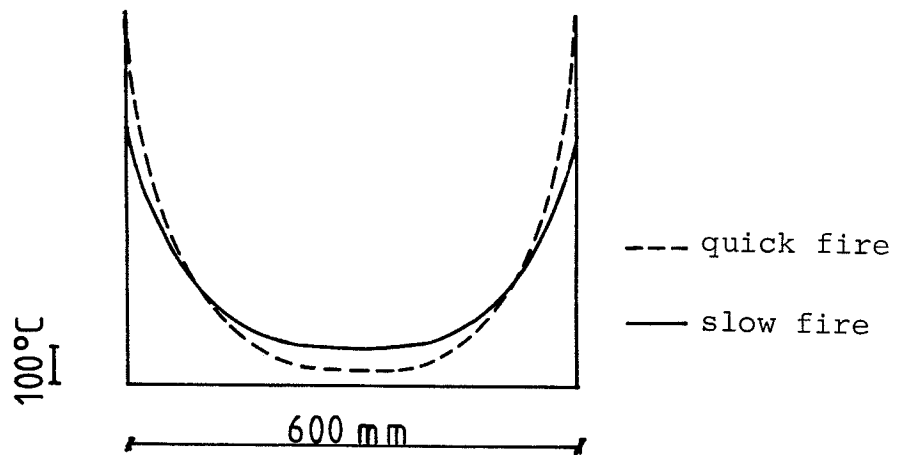be compared to the destructive potential of another
fully developed fire.

One method which operates quite precisely for many
applications is based on the concept of normalized
heat load, which has been suggested by Harmathy and
Mehaffey in a number of papers ([9], [10] and others).

The normalized heat load is defined as the integration
in time of the heat flux from the fire penetrating
the boundaries of the enclosure divided by the thermal
inertia of the boundaries, i.e. the normalized heat
load is the heat absorbed during fire exposure by the
boundaries of the enclosure per unit surface area
divided by the thermal inertia of the boundaries.

Bøhm ([11], [12]) has found that the normalized heat
load can be used for protected steel structures, but
not for unprotected ones, because these are sensitive
to the gas temperature development.

He also finds that the effect of a variation of the
materials of the compartment boundaries can be taken
into account by multiplication of the fire load as
well as the opening factor by a constant, which is
the relation between the thermal inertias of the
actual boundaries and the boundaries of a reference
compartment.

This confirms the method, which has previously been
adopted by Pettersson, Magnusson and Thor [13].

Temperature distributions of a quick and a slow fire.

However, in case of a heavy construction such as a concrete construction, the destructive potential of a fully developed fire is not only related to the integration of the heat flux, but also to the temperature -time development.

Considering a quick and a slow developing fire of the same normalized heat load, the quick fire will give rise to larger temperatures near the surface of the cross section than the slow one, and the slow fire will give rise to larger temperatures in the middle of the cross section than the quick one.

So the quick fire may be more severe than the slow one if for example the main reinforcement bars are positioned near the surface, and the slow one may have the most severe effects, if the bars are placed at the center line; and yet both fires have the same normalized heat load.
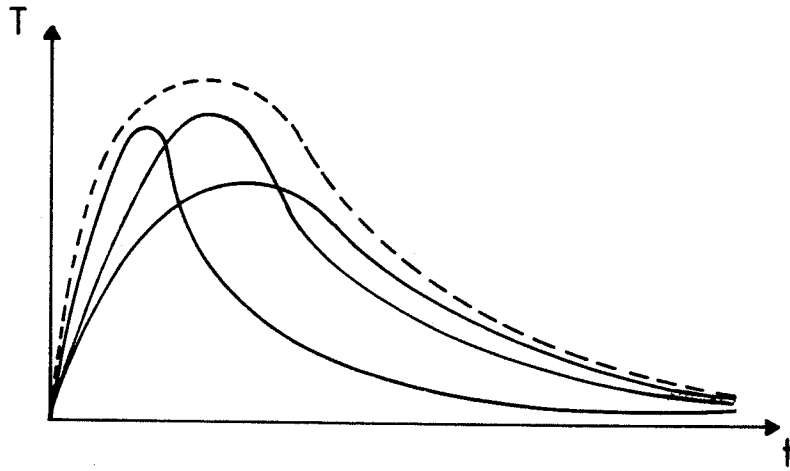
From the previous argumentation against the possibilities of establishing any equivalency between various fire courses, it can be concluded that the destructive potential of a fire on a concrete structure depends on the heating rate and on the maximum temperature as well as the duration of the fire course.

This means that at least three parameters are necessary for a description of a fully developed fire of the same destructive potential as an actual fire, and the importance of each parameter depends on the various properties of the construction.

If it is accepted that the gas temperature-time curve of the compartment is sufficiently expressing a fire exposure, still quite many shapes are possible for the fire curve.

In practice, the number of calculations and especially the number of testings, which are possible to make for the fire safety design of a construction is very limited, and if the construction should be able to withstand a great variety of fires, it is necessary to make the design for a few fires chosen in such a way that if the resistance of the construction is acceptable to these fires, it can be deduced, that it will also be acceptable to any one of the other fires.

This situation can be achieved if one ore more design fires are selected so that each of the actual fire curves are totally covered by at least one of the design curves, i.e. the temperature of the design curve is greater than or equal to the temperature of the actual curve at any time of the fully developed fire course.

Actual fire courses and a design curve.

In this way a few standard fire curves with cooling
phases could be an operational basis for a structural
fire safety design, although the design sometimes
may be fairly much on the safe side.

Using flexible light weight ovens, which can easily
follow various heating curves, the problems of fire
testing for fully developed fires seems to be limited.

By this the perhaps worst hindrance for the introduc-
tion of a design for a fully developed fire course
seems to be removed.

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