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Statically loaded structures are usually designed with respect to their construction in such a way that they can withstand any negative environmental influences such as an increased temperature stress during a fire over a period of at least some time.

For this purpose tunnel construction elements are lined on their surface with thermal blankets in order to delay the heating of the tunnel linings during a fire.

Pictures of damage caused by tunnel fires in recent years have shown that both unreinforced-concrete structures as well as reinforced-concrete structures display a certain load-carrying capacity reserves after having been subjected to fire. An adequate calculation method for proving such load-carrying capacity reserves which is based on the same principles as proof against pressure of mountain mass and other loads is not known yet.

Since the national authorities are increasingly demanding the presentment of such evidence in the course of the approval proceedings, it is the object of the present invention to develop a calculation method which works without applying complex numerical methods and makes it possible to provide a safety analysis that certifies an adequate load-carrying capacity of statically loaded concrete structures during and after a fire.

In this way it is intended to create a reliable source of information which gives information on how long a concrete structure will maintain its stability under a fire load before it collapses. This information is important in order to determine how much time is available for the evacuation of endangered persons after the start of the fire and the time at which it is no longer advisable for the fire-fighting staff to enter the structure.

This problem principally affects any statically loaded structure. As an example, however, the problem of fires in a tunnel will be discussed below in particular. Such proof of safety against collapse for tunnel systems is especially important in municipal areas where the collapse of tunnel linings can cause the ground to cave in right up to the built-up surface regions. This problem becomes even more obvious when one considers that the distance between the tunnel ceiling and the ground level may be merely a few meters, e.g. 7 to 15 meters.

In order to enable taking into account the fire loads and their effects on the load-bearing construction, three relevant sections must be distinguished in the sequence of the analysis:

- The definition of the fire load.

- The calculation of the temperature gradients in the structural component.
- The proof of the safety against collapse of the structure.

In order to define the fire load it is possible to use either predefined fire curves or fire-load curves especially calculated for the respective case, with the type of traffic or the conveyed goods being considered in this case. Said fire-load curves determine the progress of the temperature over time.

It is thus possible to determine the temperature gradients in the structural element depending on time with the thus predetermined fire loads through the solution of differential equations. Computing programs are available for the solution of this heat conductivity problem.

Respective calculations of the tunnel linings for so-called standard load cases such as dead load, loading in consequence of groundwater and of earth pressure, and living load are usually available at the beginning of each construction development. These calculations are usually performed with the help of linear bar statics (linear theory of frame analysis). The proof of the temperature stresses is also made according to this system.

At first it is necessary to determine the material parameters for the structural component under the temperature influence. In the case of a tunnel this is for reinforced concrete. For this purpose the properties during the fire and the residual strength after the fire are determined both for the concrete as well as for the steel. The precondition is that the inner armoring system is protected in a respective fashion. If the inner armoring that is subjected to the temperature is not protected from overheating, no residual strength and thus no load-carrying capacity will be maintained. The protection of the armoring can only be achieved by a respectively high covering. The prevention of the flaking of the covering is of high importance, either by reinforcing the skin or by adding polypropylene fibers.

Different material properties are obtained at every place depending on the temperature T in the cross section which is given by the temperature gradient at every time t . As a result, for every point in the cross section the valid modulus of elasticity is obtained as

$$E_m = f(T(f(t)))$$

with the modulus of elasticity being a function of the temperature and that on the other hand a function of time. The same applies for the respectively valid concrete or reinforced concrete or steel strength

$$\sigma_{m,m} = f(T(f(t))).$$

These material properties, which are always different over the cross section, must now be integrated over the cross-sectional height and lead to the overall properties of the cross section at the temperature stress prevailing at such time.

With the help of a computer program, the individual cross-sectional properties for the predefined maximum expansion states are integrated and the respectively possible bearing loads are calculated. The envelope which is formed from the variation of the edge expansions leads to the bearing load curves.

As is common practice for reinforced-concrete bearing load curves, the bearing capacity of the individual cross section depends on the definition of the maximum expansions. The increase of the permissible concrete compression in particular makes a considerable contribution. The definition of the maximum expansions has a particular influence on temperature-loaded cross sections.

As expected, the temperature stress leads in total to a reduction of the bearing capacity of the cross section and simultaneously to unsymmetrical bearing behavior of the cross section because the material properties per se for geometrically symmetrical cross sections are now unsymmetrical. As has been noticed, an increase of the maximally permissible concrete compression leads to a far from insubstantial increase of the bearing capacity of the cross section. The explanation is that although it is not possible to build up any higher effective concrete stress at the edges, higher load reserves can now be activated in the slightly cooler inner regions of the cross section at higher associated concrete compression.

Like the bearing load of the cross section, the stiffness of the cross section decreases with increasing temperature stress, which stiffness on the one hand depends on the decrease of the associated modulus of elasticity and on the other hand occurs by the continuous reduction of the cross-sectional height due to the overload by the temperature since all cross-sectional shares fail with temperature stresses of more than 700°C.

As in the determination of the bearing load, the share in the stiffness of the overall cross section can be determined at every place through the definition of the respectively valid modulus of elasticity.

As has already been mentioned, this method intends to make do with linear bar statics, i.e. finite element method. In order to ensure this, it is necessary to convert the temperature stresses of the cross section.

In linear bar statics, only linear courses of temperature gradients are permissible. As was noticed, the actual temperature gradients deviate strongly from linear progresses. As a rule, the edge zones are usually strongly temperature-loaded. Cross-sectional parts situated further inwards and the part

of the cross section averted from the fire are usually not or hardly temperature-stressed.

Similarly, the reduced material properties are relevant only in the zones of increased temperature stress. The original material properties always apply to the remaining cross section.

As in the determination of the bearing load, it is possible to define an inner state of stress with respect to each temperature stress which is determined at each place on the one hand by the respectively prevailing temperature and the expansions occurring thereby. On the other hand, the material properties can be defined for each point depending on the temperature, so that the total load can be calculated from the predetermined temperature gradient.

These internal forces can each be converted in a subsequent step into equivalent outer temperature stresses which would produce these internal force values, but not the same state of stress.

With these equivalent temperature loads it is now possible to determine the internal forces with the help of linear bar statics and their results can be delimited from the bending moment and normal force against the load curves.

With the aforementioned calculation steps all preconditions have been fulfilled to make the collapse analysis. The equivalent temperature load is applied in addition to the loads that act in any case and the internal forces are determined with the help of linear bar statics. The interaction of these internal forces is delimited against the load curves of the respective cross section and the collapse analysis is thus produced.

If the proven system is to remain stable even after the fire with a predetermined amount of security until respective repair measures are undertaken, it is also necessary to conduct a collapse analysis after the fire.

This must be performed with the material properties which are reduced accordingly by the fire load both for concrete as well as for steel. This must be considered in determining the necessary armoring in such a way that at first the internal forces in the construction must be determined without the fire load, but with altered stiffness conditions. The rating analysis is then made with the reduced material properties, with the required safety factors having to be specified.

The invention is now explained in closer detail in an embodiment by reference to the enclosed figures, wherein:

Fig. 1 shows a diagram representative of a temperature curve in the cross section of a building component depending on the time;

- Fig. 2 shows a schematic representation of an internal force curve in a tunnel cross section at the beginning of a fire;
- Fig. 3 shows a diagram according to fig. 2 180 minutes after the start of the fire;
- Fig. 4 shows a diagram in which the bending moment is entered over the normal force, and
- Fig. 5 shows a flow chart which represents the sequence of the method in accordance with the invention.

There is given a conventional, elastically embedded tunnel cross section with the following cross-sectional dimensions:

Internal formwork cross section = 40 cm

Inside and outside armoring = 5 cm² /m

Concrete quality B300

Steel quality ST 55

Modulus of subgrade reaction 100.000 kN/m²

Inside diameter approx. 6 m

Capping = 10 m light soil

Water level 2 m under upper edge of the vault

Outside concrete cover = 5 cm

Inside concrete cover = 10 cm

Inside protective armoring, d = 3 cm (is used for limiting flaking under temperature influence)

The fire load is applied constantly around the tunnel cross section. With increasing duration of the fire the temperature load increasingly penetrates the cross section:

Fig. 1 shows the temperature curve in a component such as a tunnel wall in progress over time. The horizontal axis states the distance from the exposed surface in centimeters. The vertical axis shows the temperature in degrees Celsius, with the individual curves showing the temperature progress after 3 mins., 6 mins., 9 mins., 30 mins., 60 mins., 120 mins and 180 mins after the beginning of the exposure.

Internal forces are obtained from the load cases of dead load, earth and groundwater pressure, with the installation of minimum armoring being required. Fig. 2 shows the curve of the internal forces in a tunnel profile at time $t=0$, i.e. before the load caused by the fire.

Fig. 3 shows in a similar diagram the internal forces 180 minutes from the start of the fire, with the internal forces from the temperature load now being superimposed.

After examining different times t it can be seen that one can expect a continuous increase of the normal force in the system which flattens off with increasing duration of the fire and depends on the stiffness of the bedding.

In contrast thereto, the moment load in the cross section increases at first exorbitantly and decreases after a maximum again in a respectively strong way. This can be explained by the fact that at the beginning of the fire the inner load by the temperature gradient acts in an extremely eccentric way. After a certain duration of the fire, however, the temperature load still continues to increase, but has a far less eccentric effect.

The safety analysis of the load-carrying capacity is performed with the associated load curves. The mentioned example shows that the cross section still has a safety against collapse of > 1 even after a fire load of 180 mins.

Fig. 4 shows a diagram in which the normal force is entered in kN on the horizontal axis and the bending moment kNm on the vertical axis.

It can thus be proved that without any additional measures the stability of the cross section with a safety > 1 is also ensured during the fire.

Notice must be taken in addition to the above steps that in the course of the calculation any cross-sectional parts whose temperature load goes beyond 700°C are eliminated. This corresponds to the defined material properties.

The calculation model in accordance with the invention avoids the application of complex numerical statements and allows the proof of safety against collapse of supporting systems under fire load on the basis of the elementary science of strength of materials and reinforced concrete construction by using linear bar statics.

The flow chart of fig. 5 shows the calculation procedure for the model in accordance with the invention. Based on a definition of the fire load in step 1, the temperature load of the cross section is calculated in step 2 and the calculation of the time steps t_i is performed in step 3. All time steps are then calculated in step 4 in order to enable the determination of the state after the fire in step 5. The requirements placed on the construction can be derived therefrom in step 6.

In detail, step 3 can be regarded as a subprogram in which the system stiffnesses are determined at first in step 31 in a loop and the temperature and equivalent load in step 32. The dimensioning of the components occurs in step 34 and in step 35 it is queried whether the load-carrying capacity is proven. If yes, the program returns. If not, it is returned to step 31.

With this calculation model it is possible to include the step of the load calculation with a low amount of effort, whereby this step is usually missing in planning. Measures such as heat insulation and other constructional measures can be examined with respect to its influence thereon. A purely empirical arrangement of such measures can thus be omitted. The principal arrangement of heat insulations in the form of matting and plastering or protective concrete can be offered an alternative which exclusively makes use of the principles of armored reinforced-concrete structures.

What is claimed is:

1. A method for proving the safety against collapse of load-bearing systems under fire load by calculating the changes of the material properties during the fire and the residual strength after the fire on the basis of data obtained by means of linear bar statics which were already obtained from the construction of the building component and which relate to the design of the building component for standard load cases such as dead load, loading in consequence of groundwater and of earth pressure, and living load, characterized in that input values for the calculation of the component under temperature influence are determined as follows:

- by definition of the fire load;
- by definition of a time t and the calculation of the thermal load over the entire cross section at said time t , and
- by determining the material properties under temperature influence and after the fire influence;

with

- load curves for this cross section at this time, i.e. the determination of the load-bearing capacity of the cross section by the determination of the interaction between bending moment and normal force by means of load curves,
- the conversion of the internal forces resulting from the temperature load into equivalent outer temperature loads, and
- the determination of the system stiffnesses applicable for the respectively viewed cross section, i.e. flexural strength and coefficient of elasticity,

being determined with said determined data over the cross-sectional height of the component for each of said time steps under consideration of the different non-linear material properties at each place of the cross section by integration within determined state at limit strains and the internal forces as obtained by using frame analysis are delimited against the then applicable load curves by taking into account the altered stiffnesses and applied substitute temperature load.

2. A method according to claim 1, characterized in that in the case of a negative analysis of the safety against collapse of the respective load-bearing system, the method steps as mentioned in claim 1 are repeated with

changed material properties to be determined until the desired safety against collapse has been achieved.

3. A method according to claim 2, characterized in that the material properties are altered by increasing the armoring and/or increasing the share of concrete and/or by including a plastic hinge in the calculation.
4. Application of a method according to one of the claims 1 to 3, characterized in that after the preparation of the construction sheets for erecting a statically loaded building structure the safety against collapse of said building structure in the case of a possible fire are determined on the basis of data obtained by means of bar statics, which data have already been obtained from the construction of the building component and which relate to the standard load cases such as dead load, loading in consequence of groundwater and of earth pressure, and living load and optionally the constructional features are altered to such an extent that the safety against collapse during a fire load or the required residual strength after the fire is given.