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Structural behaviour of concrete filled hollow steel sections exposed to parametric fire

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ABSTRACT

This article analyzes steel-concrete composite columns subjected to natural fire scenarios in order to verify that the possibility of structural collapse during or after the cooling phase is real. The main objectives of this study are: first, to highlight the phenomenon of delayed collapse of this type of columns during or after the cooling phase of a fire, and then analyze the influence of some determinant parameters, such as section size, tube thickness, reinforcement (ratio), concrete cover and column length. The results show that critical conditions with respect to delayed failure arise for massive sections, small values of the steel tube thickness and for columns with massive section.

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1. Introduction

The fire behavior of composite columns and especially concrete filled hollow steel sections has been studied extensively in various countries, while studying almost all essential parameters identified: section shape and dimensions (Dai and Lam, 2012), concrete filling (Dotreppe et al., 2010), reinforcement rate, steel tube thickness, column slenderness (Ana et al., 2012), thermal and mechanical properties of steel and concrete (Sangdo and Amit, 2009), and even the contact problem at the steel-concrete interface (Zhong et al., 2011). Most of this works were done under standard fire conditions (ISO). However, if the behavior of a structure is evaluated by means of a performance-based approach, a more realistic representation of the fire should be used, and we must ensure for the entire duration of fire an acceptable risk of failure under the effect of natural fire. The influence of such realistic fire scenarios in the evaluation of the fire resistance is a key issue in the performance-based approach, as presented for example by Fike and Kodur (2009) for concrete-filled hollow structural section columns. The required duration of stability may be longer than the duration of the heating phase; it may even be required that the structure survives the total duration of the fire until complete burnout.

Under real fire conditions, the stability of the structural element or the entire structure must necessarily be performed in the entire time domain by a step by-step iterative method. Some authors have been interested in the residual load bearing capacity of structural elements after exposure to fire, for example Han et al. (Han et al., 2002) for composite columns, and they even developed formulas for calculating the index of the residual strength.

However, structural failure that would occur at a later stage, may be an even greater threat because it would occur at the time of first inspection, not only by the fire brigades but also possibly by other people. Such a tragic incident occurred in Switzerland in 2004. It's also occurred this type of collapse during the cooling phase of a fire in a full-scale fire test conducted in 2008 by Wald (Wald and Kallerova, 2008) in the Czech Republic.

These events show the need to conduct detailed studies on the risk of delayed collapse of structures in natural fire. The evolution of material properties in the cooling phase must be available to perform such analysis. The authors' previous work on the subject has lead them to the conclusion that, for concentrically loaded simply supported concrete columns heated on three sides, a delayed collapse is a possible event (Jernay and Dimia,

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2011; Dimia et al., 2011). This previous work was conducted using the generic constitutive models of Eurocode for steel and for concrete, for the reason that these models are widely accepted by the scientific community and typically used for structural analyses.

This article analyzes steel-concrete composite columns subjected to natural fires in order to verify that the possibility of structural collapse during or after the cooling phase is real and, if so, what the parameters and conditions are that more likely lead to this undesirable behavior.

2. Analysis Procedure

A sequentially coupled thermo-mechanical analysis is used to carry out the numerical simulations. The nonlinear finite element software SAFIR developed at the University of Liege (Franssen, 2010) for the simulation of structures subjected to fire was used. The analysis was performed by first conducting a pure heat transfer analysis for computing the temperature field and afterwards a stress/deformation analysis for calculating the structural response. The discretization for plane sections of different shapes is possible by using triangular and/or quadrilateral elements. For each element the material can be defined separately. Any material can be analyzed, provided that its thermal and physical properties at elevated temperatures are known. The variation of material properties with temperature can be considered.

2.1. Thermal analysis

The temperature curves considered as input data were taken from the parametric fire model of Annex A (A.2 a) in Eurocode 1 pat 1-2 (EC1, 2002), that represents the action of a natural fire including the cooling phase. The factor Γ that appears in Eq. (A.2 a) was given the value of 1.0, which makes the heating phase of the time temperature curve of this natural fire model very close to the standard curve as shown in Fig. 1. A uniform temperature has been assumed over the height of the column. The temperature distribution in the sections was determined by 2D nonlinear transient analyses, an example of temperature distribution inside a typical column section is shown in Fig. 2.

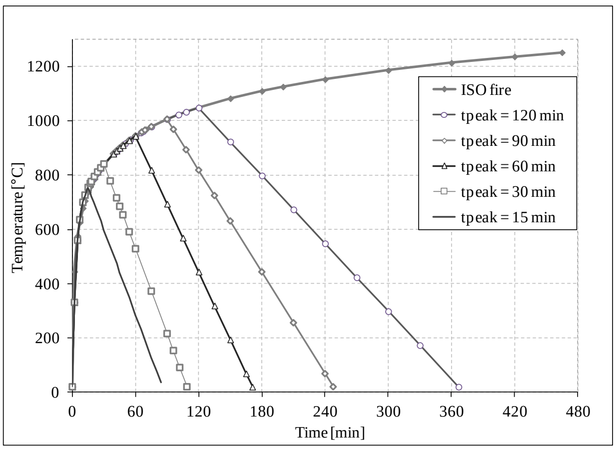


Fig. 1. Fire curves considered in the parametric analyses.

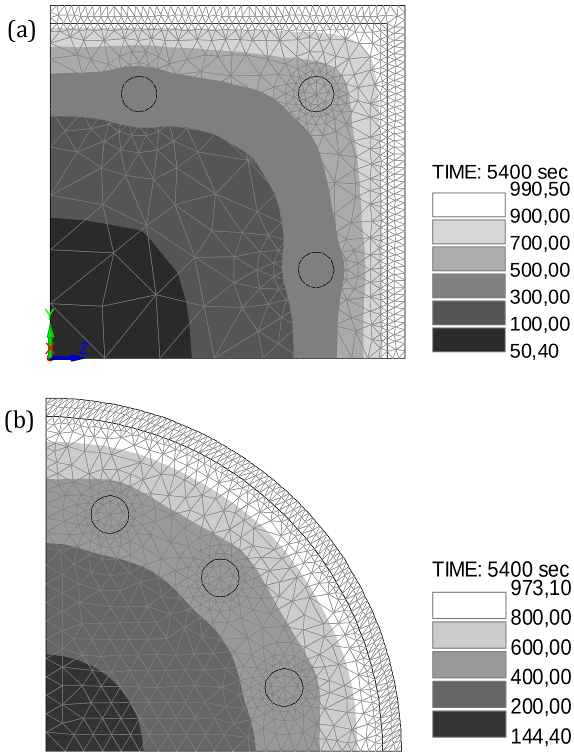


Fig. 2. Isotherms after 90 min in a section heated on four sides (1/4 modeled): (a) S400x10; (b) C300x8.

The non-steady-state 2D temperature distribution within any cross-section is determined by Fourier's equation:

$$k\left(\frac{\partial^2 T}{\partial x^2}+\frac{\partial^2 T}{\partial y^2}\right)+Q=p.c.\frac{\partial T}{\partial t}, \tag{1}$$

where k is the thermal conductivity of the material, T is the temperature, Q is the amount of heat generated in the material per unit volume, ρ is the density, c is the specific heat, t is the time, and x, y are spatial coordinates in the plane of the cross section. The temperature fields within a given section are evaluated by means of a finite element analysis in conjunction with an integration method for time steps.

2.2. Structural analysis

The columns were discretized longitudinally by means of Bernoulli beam type elements and the cross sections of the beam elements are divided into fibers that match the 2D elements of the thermal analysis.

In the model, a whole composite column is built up by means of several 2-D beam elements which are based on the following formulations and hypotheses:

- The displacement of the node line is described by the displacements of three nodes, two nodes at each end of the element supporting two translations and one rotation in plane plus one node at mid-length supporting the non-linear part of longitudinal displacement.
- The Bernoulli hypothesis is considered, i.e. plane cross sections remain plane after deformation;
- The hypothesis of Von Karman is used : the strains are small;

- The rotations are assumed to be small (note that they are evaluated in the co-rotated configuration);
- The longitudinal integrations are numerically calculated using Gauss' method;
- The integration of the longitudinal stresses and stiffness on the section is based on the fiber model;
- The mechanical interaction between the steel tube and concrete infill was modelled as perfect i.e. no slip occurs at the steel-concrete boundary
- A sinusoidal imperfection with maximum amplitude [Dotreppe et al., 2010) has been introduced in the direction of the thermal gradient (leading to bending along one single axis).
- D/th is less than $52 = \sqrt{235/f_y}$ (D: the outside dimension of the columns, th: the thickness of the steel tube) to neglect the effect of local buckling in the steel wall according to Eurocode 4-Part 1.1 (EC4, 2004).

3. Material Models at Elevated Temperatures

The numerical model takes into account the temperature-dependent thermal and mechanical properties of the materials.

3.1. Thermal models

The thermal properties of steel and concrete in the heating phase have been taken from EN1994-1-2 (EC4, 2005). This means that thermal conductivity of concrete was taken with the upper limit in the sense of EN 1992-1-2 (EC2, 2004). Siliceous concrete was chosen, with a density of 2300 kg/m^3 and a water content of 92 kg/m^3. As previously mentioned the thermal boundary conditions consist of thermal radiation and forced convection. The thermal radiation is defined by the steel emissivity which is taken equal to 0.7, whereas the forced convection is taken equal to 35 W/m^{2}K. Thermal properties of steel were considered as fully reversible during cooling. The specific mass of concrete, which decreases during heating because of the release of water, has been considered as constant during cooling. When the temperature increases in the concrete, the thermal conductivity has a tendency to decrease (EC4, 2005). It keeps the value corresponding to the maximum temperature during cooling.

3.2. Mechanical models

The mechanical properties of the steel have been considered as reversible, which means that stiffness and strength are recovered to full initial values during cooling. In the thermal elongation curve, the plateau corresponding to the phase change that occurs around 800°C at a level of 11 x 10^{-3} during heating occurs at slightly lower temperatures, around 700°C, at a level of 9 x 10^{-3} during cooling. When steel is back to ambient temperature, there is no residual thermal expansion.

For concrete, a residual thermal expansion or shrinkage has been considered when the concrete is back to ambient temperature. The value of the residual value is a function of the maximum temperature and is taken from experimental tests made by Schneider in 1979 and mentioned in (Schneider, 1985).

Compressive strength of concrete does not recover during cooling. According to EN 1994-1-2, an additional loss of 10% has been considered during cooling. This means that, for example, if the compressive strength has decreased from 1.00 to 0.50 at a given temperature, it will decrease to 0.45 when cooling back to ambient temperature. This assumption is of course the key for all the predictions presented in this paper and thus the reliability of the conclusions. In the stress-strain relationship of concrete, the strain corresponding to the peak stress during cooling is considered fixed to the value that was attained at the maximum temperature Fig. (C.2) of Eurocode 4.

4. Parametric Studies

The main parameters affecting the fire resistance of at elevated temperatures have been investigated). The parameters studied are the outside dimension of HSS columns (D), the thickness of the steel tube (th), the length of column (L), the amount of steel reinforcement (As) and the duration of the heating phase tpeak.

The values that are considered in the parametric analyses are given below:

The outside dimension of the columns (D), the amount of steel reinforcement, the duration of the heating phase does not exceed 120 minutes and the wall thickness. Concrete cover has been fixed at a minimum value of 30 mm. The relative slenderness values of the columns at room temperature were calculated in accordance with Clause 6.7.3.3 of EN 1994-1-1 (EC4, 2004) assuming hinged end conditions.

5. Influence of the Duration of the Heating Phase

Fig. 3 summarizes the analyses performed to examine the influence of the duration of the fire. Each curve is related to one of the fires shown in Fig. 1 and is the result of numerous simulations performed with different load levels. From the load bearing capacity at time t=0, here 5066 kN, the load has been progressively reduced and numerous simulations have been performed, each one for a different load, yielding a fire resistance time that increases as the load is decreased. In a real building, the columns are subjected to different loads, the level of which is not known to the fire fighters. What each curve shows is that, for the curve marked as t_{peak}=30 min for example, if a column is subjected to a load of 2000 kN, it will fail after 40 min. Each of these curves shows whether there is a potential dangerous range of the load that could lead to collapse during or after the heating phase. If this is the case, the fact that a delayed collapse will occur or not in a building will depend on the actual load level on the columns.

It is thus possible, according to this model, to have in certain cases a structural collapse some time after the end of heating phase.

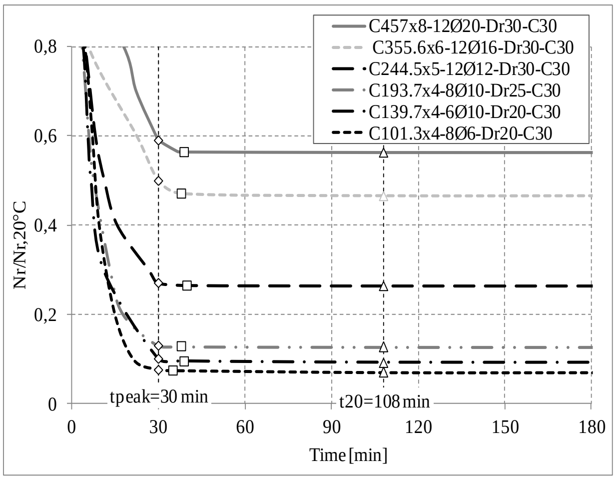
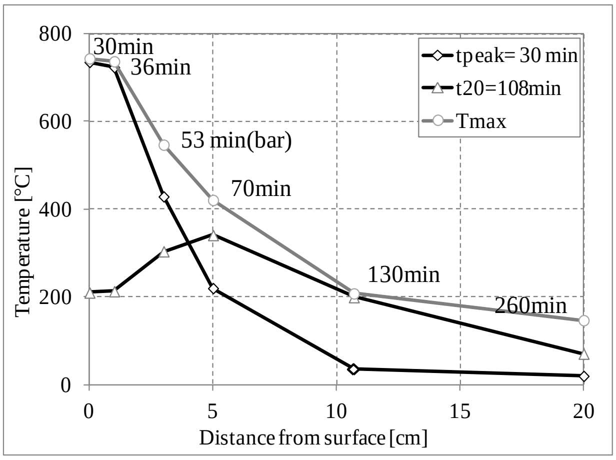


Fig. 3. An example of evolution of the load bearing capacity for a column subjected to natural fire, t_{peak}=30min.

6. Influence of the Section of the Column

The outside dimension of the columns (D) considered are: Square section: S150x4-4Ø12-Dr25-C30, S200x5-12Ø10-Dr25-C30, S300x6-12Ø16-Dr30-C30, S400x10-12Ø20-Dr30-C30 and S500x10-12Ø25-Dr30-C30. Circular section: C139.7x4-6Ø10-Dr20-C30, C193.7x4-8Ø10-Dr25-C30, C244.5x5-12Ø12-Dr30-C30, C355.6-6-12Ø16-Dr30-C30 and C457-8-12Ø20-Dr30-C30.

Each section was exposed to different fires represented on Fig. 1.The analyses have been repeated for different section dimensions of the column with a length of 4m. Fig. 4 shows the peak temperature at various depths of cross section (S400x10-12Ø20-Dr30-C30) to the parametric fires for 30 and 90 min. compared with the temperatures at the end of heating. It can be seen that the grater the depth of concrete, the grater the temperature increase during and after the cooling phase.



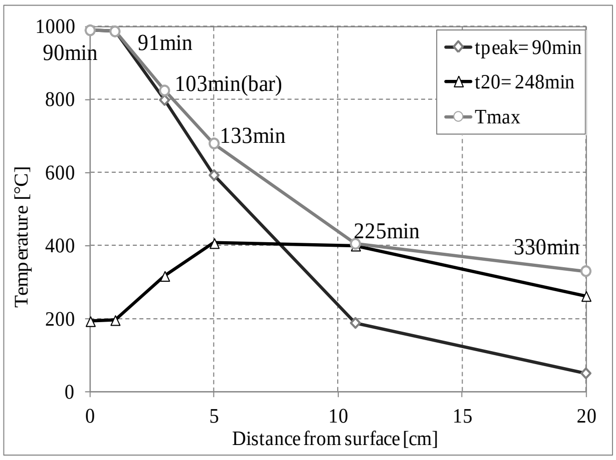


Fig. 4. Evolution of temperatures in a section S400x10-12Ø20-Dr30-C30: (a)- Evolution des T° sous t_{peak}=30min; (b) Evolution des T° sous t_{peak}=90min.

Fig. 5 summarizes the results of the analysis performed on the basic column (4m length) with different shapes for a natural fire with different duration of heating. The curve that gives the evolution of the load bearing capacity is the result of numerous simulations performed with different load levels. From the load bearing capacity at time t=0, the load has been progressively reduced and numerous simulations have been performed. The obtained relationship between the applied load and the fire resistance time gives us the evolution of the load bearing capacity of columns. The load bearing capacity continues decreasing after the maximum gas temperature is reached in the compartment. As can be seen, these curves can be considered as the resistance curves for studied columns corresponding to the lowed load witch the column would support during the fire time. The load bearing capacity continues decreasing after the maximum gas temperature is reached in the compartment.

7. Influence of the Steel Tube Thickness

For a section of 300x300 with different values of the thickness of the steel tube, a collapse, in the cooling phase exists for the majority of the columns studied, and the most critical situations in respect of this late failure arise for small thicknesses for small values of the thickness like it presented on Fig. 6. For the short fire with 30 minutes of heating, we observe the insensitivity of $t_{collapse}$ to the steel tube thickness, for example for a thickness of 4mm , collapse occurs at 12 minutes from the beginning of cooling phase. The same situation is observed for t_{peak}=60minutes, when failure occurs at about 16minutes after the start of cooling. So for short and medium fires (heating between 30 and 60 minutes), the steel tube thickness has almost no influence on the value of $t_{collapse}$. The phenomenon of delayed collapse is more pronounced for small thicknesses, and the $t_{collapse}$ decreases by increasing the thickness until closer to t_{peak}.

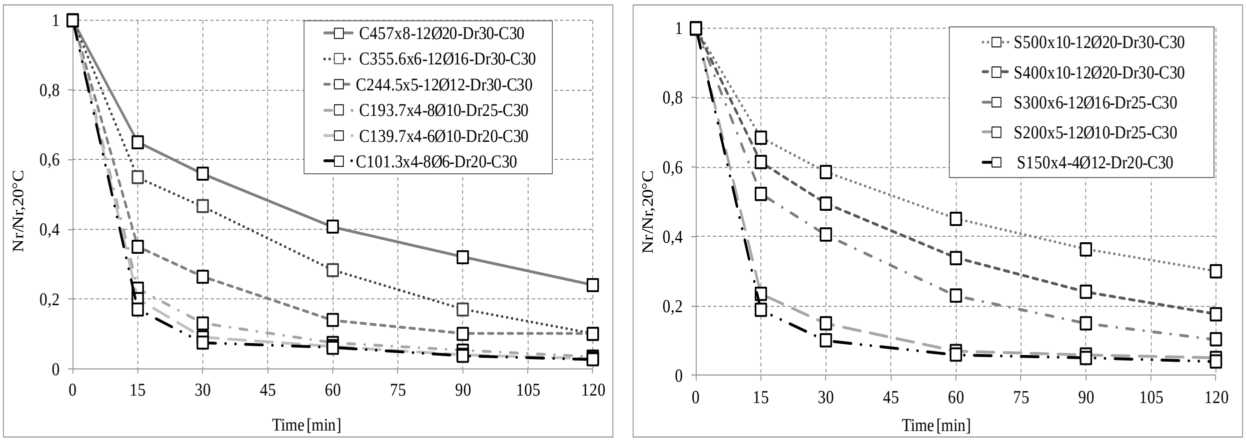


Fig. 5. Evolution of the load bearing capacity for columns subjected to natural fires.

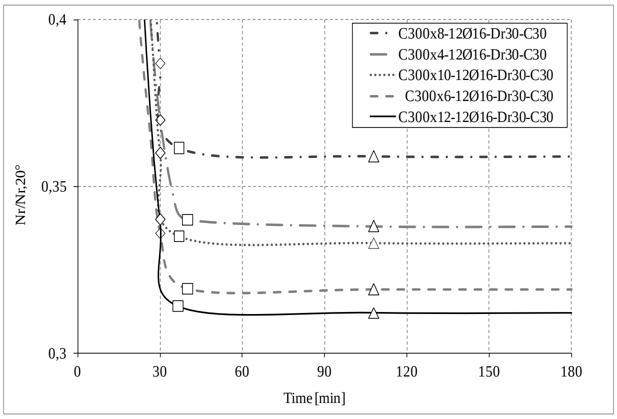


Fig. 6. Influence of the steel tube thickness, t_{peak}=30min.

This can be explained by the fact that increasing the thickness of the steel tube reduces the size of the concrete core which is element responsible for the delayed failure. So a smaller wall thickness implies a larger section of the concrete and therefore more risk of delayed collapse and vice versa.

8. Influence of Reinforcement Rate

Fig. 7 shows clearly a delayed failure for the section S260x7-Dr30 with different reinforcement rates under fire t_{peak}=30min. It occurs about 10 min after the start of the cooling phase. Increasing the amount of reinforcement slightly increases the duration of the $t_{collapse}$. The influence of reinforcement rate on the time of collapse becomes more pronounced when the heating time increases to 60 and 90 minutes. When t_{peak} rises to 90 minutes, increasing the reinforcement section increase the $t_{collapse}$ of an average of about 1 minute/1%, until at As = 3.8% a jump in the value of $t_{collapse}$ of the order of 7 minutes compared with the previous value.

Arrive at a percentage of 7.5%, the value of $t_{collapse}$ becomes constant and insensitive to the change of reinforcement. So increasing the reinforcement section increases the concrete resistance and thereby retards columns failure. However, for a heating of two hours, the delayed failure exists 35 minutes after the start of cooling, but the reinforcement rate will have almost no influence on the duration of fire resistance of the element. Fig. 8 shows the fire resistance curve of columns depending on the reinforcement rate.

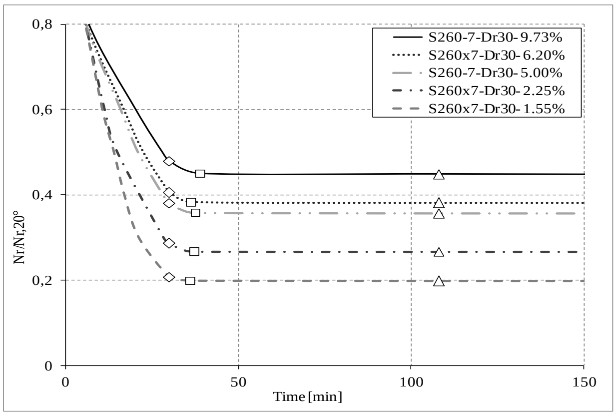


Fig. 7. Influence of the reinforcement rate.

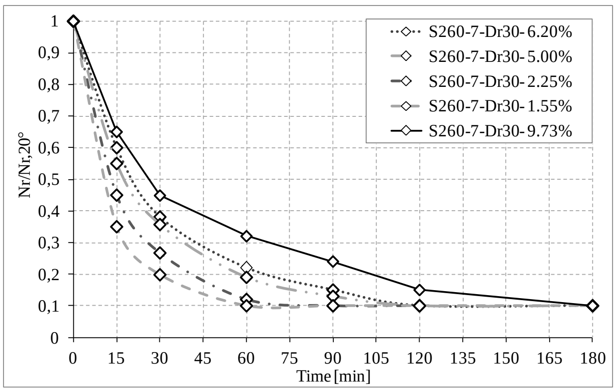


Fig. 8. Fire resistance curves for a column S260x7-Dr30 with different reinforcement rates.

9. Conclusions

A numerical model has been used in this study for concentrically loaded composite steel-concrete columns heated on four sides to highlight the phenomenon of delayed collapse of this type of columns during or after the cooling phase of a fire. The results show that a failure during the cooling phase of a fire is a possible event even after several minutes after the end of the heating phase.

The main mechanisms for these delayed failures are to be found in the fact that temperatures in the central zones of the concrete core can keep on increasing even after the gas temperature is back to ambient and also in the fact that concrete may lose additional strength during cooling compared to the situation at maximum temperature.

Parametric studies were carried out by means of this numerical model in order to investigate the main parameters affecting directly this delayed collapse of composite columns under natural fire.

It has been shown that the most critical situations with respect to delayed failure arise for short fires, for columns with massive section, for columns with small values of the thickness of the steel tube and for concrete cores with important reinforcement rate.

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