Effect of local instability on fire response of steel beams

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Abstract

Purpose – This purpose of this paper is to quantify the effect of local instability arising from high shear loading on response of steel girders subjected to fire conditions.

Design/methodology/approach – A three-dimensional nonlinear finite element model able to evaluate behavior of fire-exposed steel girders is developed. This model, is capable of predicting fire response of steel girders taking into consideration flexural, shear and deflection limit states.

Findings – Results obtained from numerical studies show that shear capacity can degrade at a higher pace than flexural capacity under certain loading scenarios, and hence, failure can result from shear effects prior to attaining failure in flexural mode.

Originality/value – The developed model is unique and provides valuable insight (and information) to the fire response of typical hot-rolled steel girder subjected to high shear loading.

Keywords Civil engineering, Structural engineering, Structural fire engineering and diagnostics

Paper type Research paper

Introduction

Structural members, when exposed to fire, experience loss of capacity and stiffness due to temperature-induced degradation in strength and modulus properties of constituent materials. When the capacity (typically moment capacity) at the critical section of the member drops below the applied moment due to loading, failure occurs. The time to reach this failure is referred to as fire resistance. In contrast to ambient temperature design philosophy, where a beam is generally designed to satisfy flexural limits state, and then checked for shear resistance, failure of beams under fire conditions is derived based on flexural limit state only. This flexural limit state, although valid for most common scenarios, may not be representative in certain situations where shear effects can be dominant in a fire-exposed member.

The most common example where shear forces can be dominant in beams is concentrated loads acting near end of beams connecting to offset columns in buildings (Hall, 1954). Shear can also control the design of transfer girders, coped beams, short span beams and deep beams/

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plate girders. In general, webs in W-shaped sections or plate girders can be thin compared with that of flanges. These thin webs experience rapid rise of temperature under fire exposure as compared with flanges. Therefore, shear capacity of steel beams can degrade at a much higher rate than flexural capacity. Temperature-induced loss of strength and modulus properties of steel in the web can cause severe instabilities at material and sectional levels and can lead to onset of global instability (Kodur and Naser, 2014; Naser and Kodur, 2015; 2017).

A review of literature clearly shows that most previous studies focused on fire behavior of beams predominantly subjected to bending effects (Kodur and Dwaikat, 2010; Dwaikat and Kodur, 2011; Aziz and Kodur, 2013). These studies considered effects of various factors on flexural response of fire-exposed steel beams, such as restraint conditions and inelastic response. However, the effect of shear parameters on fire response of beams was not considered. To evaluate effect of shear on response of a fire-exposed steel girder, a numerical study is carried out using a three-dimensional nonlinear finite element model. The developed model can trace the fire response of hot-rolled W-shaped beams and girders subjected to significant bending moment and shear force. The model was validated against test data on beams and then the model was applied to examine the influence of shear on fire response of steel girders under different loading configurations and web slenderness.

Numerical model

The three-dimensional finite element model of the beam has geometry of a typical hot-rolled steel W-section commonly used in flexural members. This model was developed in ANSYS and accounts for several parameters including geometric and material nonlinearities, temperature-dependent material properties and various failure limit states. For undertaking fire resistance analysis, the beam is discretized with different thermal and structural element, available in ANSYS. SOLID70 and SURF152 elements are used as thermal elements to simulate heat transfer to the beam under fire exposure. SOLID185 is also used for modeling the structural response of three-dimensional solid structure (ANSYS, 2011). Figure 1 shows typical steel beam and associated finite element model.

For undertaking fire resistance analysis, temperature-dependent thermal and mechanical properties of steel are to be input to the finite element model. These thermal and mechanical properties of structural steel are assumed to vary with temperature as per Eurocode 3 recommended relations [European Committee for Standardization (CEN), 2005]. To simulate the response of fire-exposed steel girders, two stages of analysis are to be carried out at each time step. The first stage examines heat transfer between fire source and steel girder. Then, cross-sectional temperatures are input to the second stage of simulation to carry out structural analysis. In the structural analysis, both temperature and loading is applied simultaneously and the mechanical behavior is evaluated. Sectional capacity can be obtained as well as shown in an earlier study (Kodur and Naser, 2014).

Flexural and shear failure occur once the bending moment (or shear force) due to applied loading exceed the moment (or shear) capacity at a critical section. Also to check failure, mid-span deflection is compared against deflection limit state used in BS 476 (British Standard Institute, 1987). The beam is said to fail when the beam attains a deflection of (L/20) or rate of deflection reaches (L2/9000d), where L and d are the span and depth of the beam, respectively.

Validation of numerical model

The above finite element model was validated using data from tests on conventional steel beams. Kodur and Fike (2009) reported detailed results from fire resistance test on a W12 × 16 A992 steel beam exposed to ASTM E119 standard fire. The beam was insulated with 50-mm thick spray-applied vermiculite-based fire insulation to achieve a 2-h fire resistance

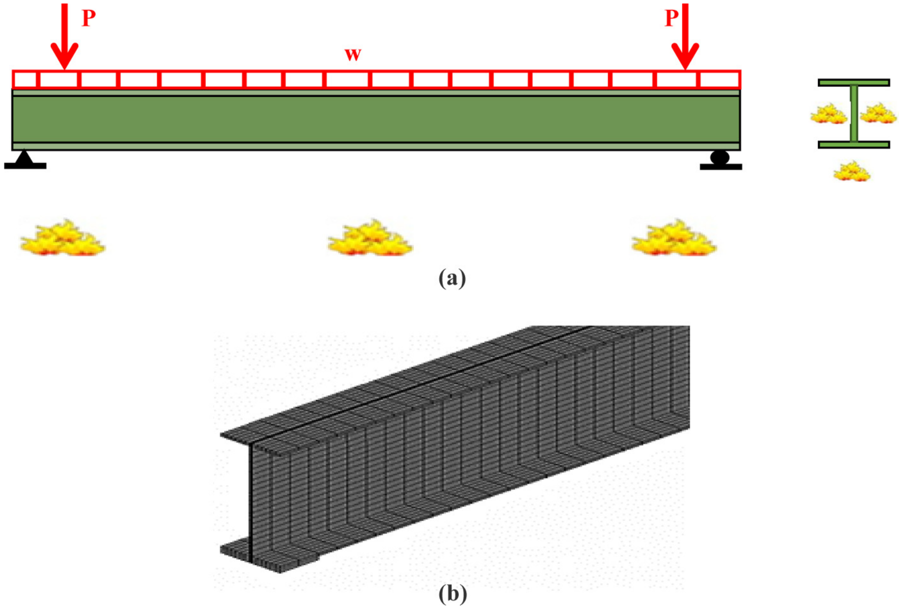

Figure 1.
Typical steel beam and developed finite element model: (a) typical beam loaded with high shear forces; (b) isometric view of the finite element model

rating. The beam was loaded with two symmetrical point loads 1.5 m away from end supports. This loading represents 31 and 5 per cent of its room temperature flexural and shear capacity, respectively, as per AISC provisions (2011).

The tested beam is analyzed using the above developed model. The various output parameters generated in the analysis, i.e. cross-sectional temperature profile, mid-span deflection and failure mode, are compared against measured data from fire test. Figure 2(a) shows a comparison of measured and predicted temperature (average of both flanges and web) in the steel beam as a function of fire exposure time. As can be seen, there is a good agreement between predicted and measured temperatures up to the first 45 min. At 45 min, average temperature in steel section was around 350°C. Beyond 45 min, the predicted steel temperatures (from model) tend to be slightly higher than the measured ones in temperature range of 350-600°C. This can be attributed to differences in assumed and actual thermal properties of fire insulation at elevated temperatures.

A comparison of predicted and measured mid-span deflection response of the tested steel beam is shown in Figure 2(b). The beam undergoes only small deflection in initial stages of fire exposure and this remains constant till about 90 min. This can be attributed to the factor that steel does not experience significant degradation in strength in 20-400°C but experiences moderate loss in elastic modulus in this temperature range. However, as the temperature in steel beam reaches 550°C, at about 100 min, strength and stiffness properties of steel start to rapidly degrade which results in sudden increase in deflection. After 120 min of fire exposure, steel loses most of its strength and stiffness as the temperature of the beam rises to 600°C, which produces runaway failure in the beam at 122 min.

Figure 2(c) and (d) show the degradation of moment capacity at mid-span section and shear capacity at support section with fire exposure time, respectively. The moment and shear capacity reserve at critical sections of beam, at the start of fire exposure, is about 3.3 and 20

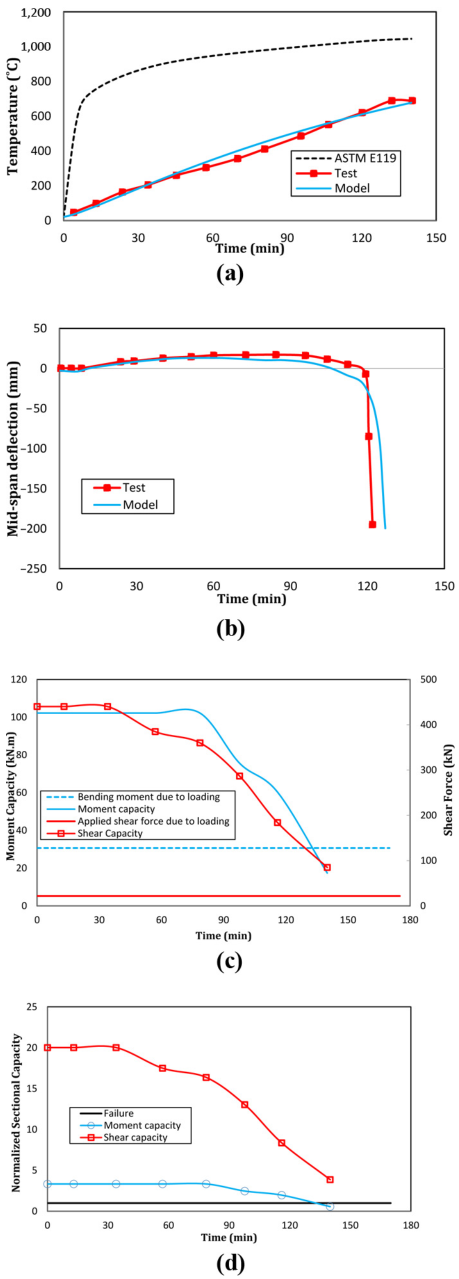

Figure 2.
Comparison of predicted and measured parameters as a function of fire exposure time: (a) thermal response; (b) structural response; (c) degradation of moment and shear capacity at fire conditions; (d) reserve moment and shear capacity degradation at fire conditions

times that due to applied loading. This indicates that the beam has much higher reserve shear capacity than the reserve moment capacity. The moment capacity in the beam remains intact for the first 75 min due to lower average temperature in flanges (much below 350°C) of steel beam. However, shear capacity starts to degrade at 35 min due to relatively faster rise in web temperature generated from higher web slenderness. After this, steel temperature in flanges and web rise beyond 350°C. Then, both moment and shear capacity gradually degrade with increase in temperature in steel section. Degradation of both moment and shear capacity at critical sections continues till the beam fails at 130 min. Due to higher reserve shear capacity (near end supports), the beam does not experience shear failure. Failure of this beam occurred at 122 min in fire test indicating reasonable agreement with predictions from model. The good comparison on predicted temperature and deflection, as well as failure mode, with test data indicated that the proposed model is capable of tracing the fire response of steel beams.

Case studies

The above validated finite element model was applied to study the effect of shear parameters on the fire response in steel beams. The effect of loading pattern and web slenderness on shear capacity and fire response of beams is studied herein.

Effect of loading pattern

For numerical analysis, a simply supported beam of 9 m span and made of W21 × 44 section (AISC, 2010) is selected. W21 × 44 section has a flange width of 165.1 mm and overall depth of 525.8 mm. The flange and web thicknesses are 11.43 mm and 8.89 mm, respectively. The beam is made of Grade 345 MPa steel and has continuous lateral support along its 9 m span. To illustrate the effect of shear arising from different loading patterns, two configurations of this beam, "Beam 1" and "Beam 2", were analyzed.

For fire resistance analysis, "Beam 1" is subjected to uniformly distributed loading (UDL) of 20 kN/m, while "Beam 2" is subjected to UDL of 11.5 kN/m together with two concentrated loads of 320 kN applied at 0.3 m from end supports. This loading scheme generates same magnitude of peak bending moment at the critical section (mid-span); however, shear force distribution along these beams would be different. These selected load levels represent about 40-50 per cent of flexural and shear capacity at room temperature, which is similar to load levels encountered during fire conditions. The beams were designed as per AISC (2010) provisions and have room temperature flexural and shear capacity of 539 kN-m and of 966 kN, respectively. It should be noted that the loading on "Beam 2" was chosen to simulate a pure shearing state and this load setup is similar to the one used by Basler *et al.* (1960) to study shear response of steel beams at room temperature.

The above two beams were analyzed using the above developed model by subjecting them to combined loading and ASTM E119 standard fire exposure. Figure 3(a) shows temperature progression in the two beams with fire exposure time. Since these steel beams have same geometric and material properties and subjected to same ASTM E119 fire exposure, temperature rise in these beams is identical. It can be seen from Figure 3(a) that average temperatures in flanges and web reaches 435°C and 525°C at 10 min and 680°C and 770°C at 20 min into fire exposure. The different rate of temperature rise in flanges and web influences the rate at which moment and shear capacity degrade with fire exposure time.

At the start of fire exposure, Beams 1 and 2 have reserve moment capacity of 2.70 of that of the applied bending moment. However due to different applied loading patterns, Beams 1 and 2 have reserve shear capacity of 10.6 and 2.59 from that of applied shear force, respectively [as shown in Figure 3(b) and (c)]. It is clear that "Beam 1" has much higher

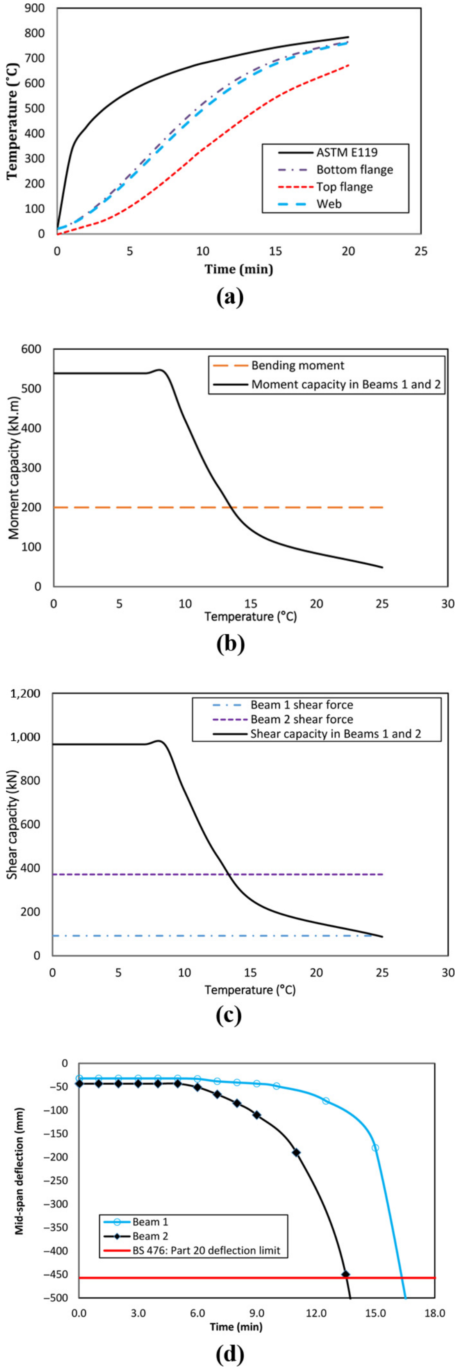

Figure 3. Thermal and mechanical response of Beams 1 and 2 with fire exposure time: (a) temperature in steel Beams 1 and 2; (b) moment capacity degradation; (c) shear capacity degradation; (d) variation of mid-span deflection

reserve shear capacity than that of "Beam 2"; hence, these beams experience failure in different modes as explained below.

Figure 3(b) and (c) show the degradation of moment and shear capacity, along with bending moment and shear force generated due to applied loading. These plotted moment and shear capacities are at critical sections, namely, mid-span for moment and location of point loading for shear force. Moment and shear capacity in both beams starts to degrade only after 8 min into fire exposure. This is due to temperature in lower flange and web crossing 400°C at that point in time as shown in Figure 3(a). As the temperature rise continues in web and flanges, further degradation of moment and shear capacity takes place until failure occurs in the beams. Furthermore, Figure 3(d) shows the predicted mid-span deflection in these three beams as a function of fire exposure time. The mid-span deflections remain small for about 10 and 6 min in Beams 1 and 2, respectively. Then, deflections increase at a rapid pace leading to runaway failure in these two beams.

As the temperature rise continues in web, further degradation in strength contribution from web occurs. This degradation of strength once reaches reduced yield strength of steel, sectional instabilities set in in the web proper to flanges. This instability can further decrease shear capacity and accelerate failure of beams via occurrence of web local buckling.

The above-generated results were utilized to evaluate failure of beams under different limit states. Failure of the beam is said to occur when moment (or shear) capacity drops below applied bending moment (or shear force) or when mid-span deflection exceeds limiting deflection criterion. It is clear from Figure 3(b) that "Beam 1" attains failure in flexural mode at about 14 min when the moment capacity drops below the applied moment due to loading (UDL). On the other hand, "Beam 2" fails in shear limiting state at 13 min, prior to onset flexural limiting state. It can be seen from Figure 3(d) that runaway (large deflection) failure occurs at 17 and 14 min in Beams 1 and 2 due to significant degradation of stiffness resulting from temperatures in steel exceeding 550°C. While "Beams 1" fails in flexural (moment) mode, "Beam 2" fails in shear limit state earlier to reaching deflection or flexural capacity limit states. Although the applied loading on these two beams resulted in similar bending moment, different loading pattern led to different shear response and failure modes. Thus, loading pattern can significantly affect the fire response of steel beams. Table I summarizes failure time in these beams. Additional studies based on different loading pattern can be found elsewhere (Kodur and Naser, 2014; Naser and Kodur, 2017; Naser, 2016).

Effect of web slenderness

Generally, slenderness of web has significant influence on shear capacity of the beam. For optimum design, slenderness of web is much higher than that of flanges, and hence, web slenderness is a critical factor in determining shear capacity in a steel beam. The effect of web slenderness on shear capacity is studied by analyzing two fire-exposed beams with varying web slenderness. "Beam 3" is a replicate of "Beam 2" shown above, but with thinner web thickness. These two beams (Beams 2 and 3) were subjected to ASTM E119 fire as well as gravity loading and were analyzed with the above developed model.

Table I.
Failure time of Beams 1 and 2

Beam	Loading	Failure time (min) Shear	Flexure	Deflection	Failure mode
Beam 1	Uniformly distributed loading (UDL)	24	14	16	Flexure
Beam 2	Concentrated loading near supports + UDL	13	14	15	Shear

Both beams have similar flange slenderness ratio of 7.22, while web slenderness ratio for "Beam 2" and "Beam 3" are 59 and 100, respectively. To illustrate the effect of web slenderness on temperature rise, predicted temperature in the web of "Beam 2" and "Beam 3" are plotted in Figure 4(a) as a function of fire exposure time. It can be seen that overall thermal response in "Beam 3" follows a similar trend to that of "Beam 2", but temperature in

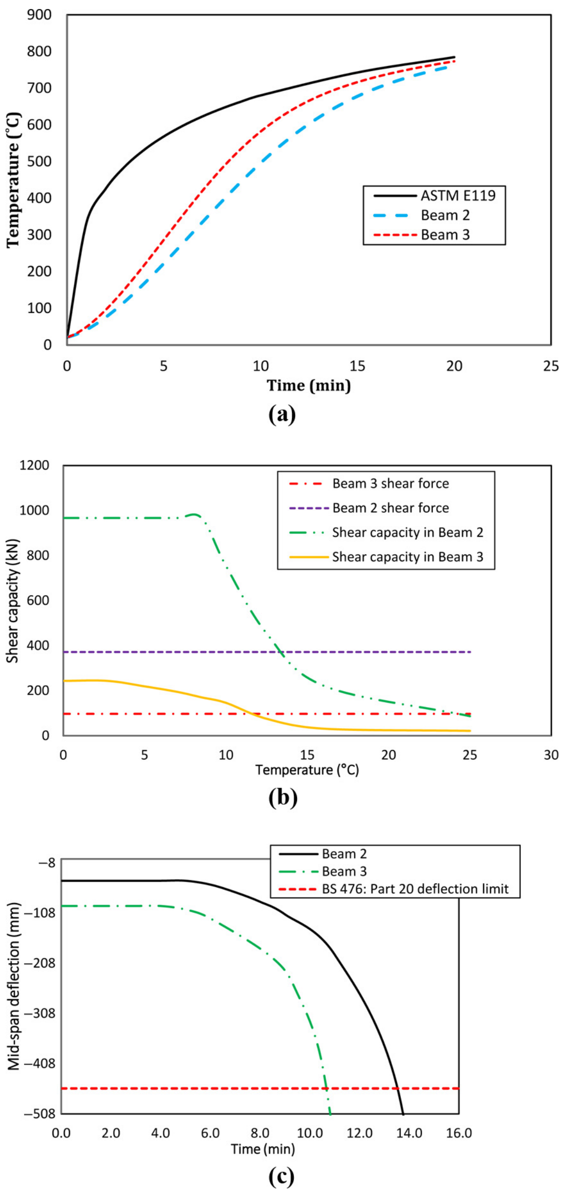

Figure 4. Thermal and mechanical response of Beams 2 and 3 with fire exposure time: (a) temperature in steel web of Beams 2 and 3; (b) shear capacity degradation; (c) mid-span deflection

web of “Beams 3” increases at a much faster pace due to slender web. Thus, faster degradation of strength and stiffness properties of web (and thus of beam) occurs in “Beam 3” as compared with that in “Beam 2”.

Since both Beams 2 and 3 were subjected to applied shear loading of 40-50 per cent of the capacity with minimum bending moment effects, these beams are likely to fail in shear. Thus, Figure 4(b) only shows degradation of shear capacity as a function of fire exposure time. Since shear capacity is mainly governed by the size of the web, shear capacity at ambient conditions of “Beam 3” is much lower than that of “Beam 2” due to higher web slenderness (reduced web thickness). When exposed to fire, moment and shear capacity of Beams 2 and 3 start to degrade after about 8.5 and 4 min of fire exposure time, respectively. At this point, internal stress (due to applied loading) reaches reduced yield strength of steel, sectional instability occurs and also plastic deformation starts to accumulate. These deformations initiate sectional instability.

Figure 4(c) compares predicted mid-span deflection in Beams 2 and 3. In general, mid-span deflections are small in the initial stage of fire exposure and then increase gradually with fire exposure time. The deflections increase at a rapid pace toward final stage of exposure due to very high temperature in steel, and this leads to failure of beams. As expected, “Beam 3”, with higher web slenderness, undergoes larger initial deflections as compared with that of “Beam 2”.

Beam 3 experiences failure through flexural and shear limit states at 12 and 11 min, respectively, as compared with 14 and 13 min in Beam 2. Table II summarizes failure modes in these two beams analyzed with different web slenderness. These results clearly infer that web slenderness influences failure mode in fire-exposed steel beams and can lead to shear failure prior to reaching flexural or deflection limit states.

It should be noted that effect of different failure modes and corresponding failure times can be more apparent in fire-insulated beams as discussed in Kodur and Naser (2014) and Naser and Kodur (2016) which have illustrated that a steel girder insulated with 1-h fire-rated insulation system will fail in 65 and 55 min due to flexural and shear effects, respectively. Therefore, accounting for shear effects can significantly alter failure times of girders in certain situations.

Conclusions

Based on the results of the analysis presented herein, the following conclusions can be drawn:

- the developed finite element model is capable of predicting fire response of steel beams where flexural or shear effects dominate the behavior of steel beams;
- in a fire-exposed steel beam, sectional instabilities can occur in web due to shear parameters prior to that in flange due to flexural parameters under certain loading and sectional configurations; and
- in fire-exposed steel beams with higher slender webs, shear capacity can degrade at a higher pace than that of moment capacity. In such beams, failure can occur in shear limit state rather than flexural or deflection limit states.

Table II.
Failure in beams with different web slenderness

Beam	Web slenderness	Flange slenderness	Failure time (min) Shear	Flexure	Deflection	Failure mode
Beam 1	59	7.22	13	14	15	Shear
Beam 3	75	7.22	10	12	10.5	Shear

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Further reading

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