Brazilian standards for steel structures fire design

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Abstract

The aim of this work is to introduce to the international scientific community the recently released Brazilian Standards for the design of steel structures under fire situations. They are: “Steel Structures Fire Design” (NBR 14323/1999) and “Fire Resistance Requirements for Building Construction Elements” (NBR 14432/2000). The aim of structure fire design is to prevent the structure from collapsing, allowing enough time for safe evacuation of building occupants, safe fire fighting operations and reduction in damage to neighboring construction. NBR-14323 provides a simplified method for calculation of fire resistance design for steel structural elements. NBR-14432 presents prescriptive recommendations for minimum fire resistance time of construction elements. This Standard also waives fire resistance time requirements depending on building height and area, fire protection devices and occupancy. They also allow the use of alternative methods of fire safety engineering. © 2002 Elsevier Science Ltd. All rights reserved.

1. Introduction

Most of the deaths, in a building fire, occur by asphyxia in the early stages. European research [1] shows that the risk of this kind of death is 30 times smaller than that in transport systems. Life risk due to structural failure from fire is even smaller.

Despite the low risk of death by fire, protection of human life must always be considered in building design. The inclusion of fire prevention and extinguishing measures and ways of allowing rapid evacuation from burning buildings must be consciously analyzed by both designer and owner, taking into account the specific

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conditions of the building, such as: size, population, occupancy, public authorities requirements and technical standards recommendations for design and equipment specification.

Escape routes, including emergency stairs, must be well marked, unobstructed and structurally safe. In order to preserve human life, the safety of structures exposed to fire in exit routes must be guaranteed during evacuation.

Structural fire safety checking can be waived for buildings from which escape is straightforward, such as small buildings or single-story buildings, except when proper preservation is desired.

Large buildings, where evacuation time is difficult to evaluate and where, generally, the exit routes are an integral part of the main structure, must have their structural safety checked. Structural fire safety, either for life or building protection, must be checked to ensure that structural collapse is prevented, thus allowing evacuation, repair and refurbishment.

High temperature exposure of structural materials, such as concrete or steel, reduces their strength and rigidity and may lead to structural collapse.

Critical temperature is the temperature that causes collapse. The true critical temperature could be determined by tests for each structural element, but, this is not economically feasible. Ordinarily, one fixes an arbitrary conventional value of critical temperature. A value based on recommendations set by Technical Standards or Codes, which is on the safe side of the critical temperature is estimated.

Structural safety is assured when steel temperature in fire situation reaches a value less than the structure critical temperature or, in other words, structural safety is verified if the design value of the effect of the actions is lower than the design value of each structural element resistance.

2. The Brazilian Standard “fire-resistance requirements for building construction elements”

In 1996, a study group created by Associação Brasileira de Normas Técnicas (Brazilian Association of Technical Standards) and consisting of professors from Universidade de São Paulo, Universidade Federal de Minas Gerais and Universidade Federal de Ouro Preto presented a draft and, in 1999, two standards were approved: NBR 14432 “Fire Resistance Requirements for Building Construction Elements [2]” and NBR 14323 “Steel Structures Fire Design” [3].

The required performance for construction elements, i.e., structural (concrete, timber or steel) or compartmentation elements, according to Brazilian Standard NBR 14432 are

- to prevent structural collapse, making the safe escape of users possible;
- to reduce damage to neighboring property; and
- to allow prompt access by the fire brigade, whenever necessary.

The Standard supplies prescriptive recommendations on the required time for fire resistance under standard fire (ISO 834 [4]). A summary is presented in Table 1.
However, it allows, alternatively, the use of any scientifically confirmed or standardized method, such as equivalent time, risk analysis (Gretener (SIA 81 [5]), for example) or more advanced methods of fire engineering. The Brazilian Standard recognizes the small chances of accidents in small buildings in which evacuation is simple, such as one-story or small-area structures and allows that some buildings, depending on their dimensions, fire protection devices and fire load be exempted from structural checking for fire situations. A summary is presented in Table 2. A list of fire loads, based on SIA 81 [5] with adaptation to Brazilian conditions, is also presented by the Standard.

### 3. The Brazilian Standard “steel structures fire design”

#### 3.1. Combination of actions (accidental load combination)

Design values for the effect of actions should be determined by combining the values of actions as follows:

\[
    F_d = \sum_i \gamma_{gi} F_{Gi,k} + \gamma_q F_{Q,exc} + \gamma_q \sum_j \psi_j F_{Q,j,k},
\]

where \( F_d \) is the design value of action on fire situation, \( F_{Gi,k} \) the characteristic value of permanent action \( i \), \( F_{Q,j,k} \) the characteristic value of variable action \( j \), \( F_{Q,exc} \) the characteristic value of thermal (exceptional, accidental) action, \( \gamma_{gi} \) the partial safety
factor for permanent action \( i \), \( \gamma_q \) the partial safety factor for variable action, and \( \psi_j F_{Q,j,k} \) the quasi-permanent value of variable action \( j \).

In fire:

\[ \Psi = 0.2 \] for places where there is neither predominance of heavy equipment that remains fixed for long periods of time, nor a large number of people.

\[ \psi = 0.4 \] for places where there is predominance of heavy equipment that remains fixed for long periods of time, or a large number of people.

\[ \psi = 0.6 \] for libraries, archives, stores, and car parks.

\[ \psi = 0.0 \] for wind loads.

\[ \gamma_g = 1.1 \] for unfavorable permanent action with small variability.

\[ \gamma_g = 1.2 \] for unfavorable permanent action with great variability.

\[ \gamma_g = 1.0 \] for favorable permanent action with small variability.

\[ \gamma_g = 0.9 \] for favorable permanent action with great variability.

\[ \gamma_q = 1.0 \].

The effect of thermal actions is taken into account by means of yield strength and modulus of elasticity reduction factors \( (k_{y,0} \text{ and } k_{E,0}) \) respectively presented in Table 3 and, occasionally, the indirect actions from restrictions to the thermal deformations. The effect of thermal expansion can be neglected if a fire-standard model is used (based on the Eurocodes 3 and 4 [6,7] recommendations and the author’s studies [8,9]). The effect of deformations due to the thermal gradient across

<table>
<thead>
<tr>
<th>Area (m²)</th>
<th>Occupation</th>
<th>Density fire load (MJ/m²)</th>
<th>Height</th>
<th>Fire protection device^a</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤750</td>
<td>Any</td>
<td>Any</td>
<td>Any</td>
<td>Minimum</td>
</tr>
<tr>
<td>≤1500</td>
<td>Any</td>
<td>≤1000</td>
<td>≤2 floors</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Stadiums, airports, railway stations</td>
<td>Any</td>
<td>≤23 m</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Open parking^b</td>
<td>Any</td>
<td>≤30 m</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Stores^c</td>
<td>Incombustible</td>
<td>≤30 m</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Any</td>
<td>≤500</td>
<td>One-story</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Industrial^d</td>
<td>≤1200</td>
<td>One-story</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Stores^d</td>
<td>≤2000</td>
<td>One-story</td>
<td>Minimum</td>
</tr>
<tr>
<td>Any</td>
<td>Any</td>
<td>Any</td>
<td>One-story</td>
<td>Sprinklers^e</td>
</tr>
<tr>
<td>≤5000</td>
<td>Any</td>
<td>Any</td>
<td>One-story</td>
<td>Two façades of fire man access^f</td>
</tr>
</tbody>
</table>

^a Minimum by law.
^b Concrete structures or steel structures but with composed beams and minimum section factors (250 m^{-1} for column and 350 m^{-1} for beams).
^c Concrete or steel structures.
^d Compartimentation as others Brazilian Standards.
^e As other Brazilian Standards.
^f Façades perimeter ≥ 50% building perimeter.
the depth of an element must be taken into account, but the linear analysis using the temperature effect reduced modulus of elasticity is allowed.

The bars of a bracing system should be designed by the following combination of actions:

\[ F_d = \sum \gamma_{g_i} F_{G_i,k} + \gamma_q F_{Q,exc} + \gamma_q 0.5F_{W,k}, \]

where \( F_{W,k} \) is the characteristic value of wind load.

### 3.2. Design yield strength on fire

The design value of yield strength in fire is determined by

\[ f_{yd,fi} = \frac{f_{yk}}{\gamma_{a,fi}}, \]

where \( f_{yd,fi} \) is the design value of yield strength in fire, \( f_{yk} \) or \( f_y \) the characteristic value of yield strength, \( \gamma_{a,fi} = 1 \) or 0, the partial safety factor for steel in fire.

To comply with the Brazilian Standard, any scientifically confirmed or standardized method, is allowed. As an option, it presents a simplified method that calculates the fire design resistance of steel structural elements (summary in Section 4.3.1) and composite steel–concrete structures (beams, columns and slabs) based on Eurocodes 3 and 4 [6,7], adapted [8] to the Brazilian structure standards.

According to “Steel Structures Fire Design” Standard, the temperature of steel structural elements can be calculated by analytical methods based on heat transfer theory (formulae supplied by the Standard) or experimental analysis.

<table>
<thead>
<tr>
<th>( \theta_a ) (°C)</th>
<th>( k_{\gamma,\theta} = f_{\gamma,\theta}/f_{\gamma} )</th>
<th>( k_{E,\theta} = E_{\theta}/E )</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>100</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>200</td>
<td>1.000</td>
<td>0.900</td>
</tr>
<tr>
<td>300</td>
<td>1.000</td>
<td>0.800</td>
</tr>
<tr>
<td>400</td>
<td>1.000</td>
<td>0.700</td>
</tr>
<tr>
<td>500</td>
<td>0.780</td>
<td>0.600</td>
</tr>
<tr>
<td>600</td>
<td>0.470</td>
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<tr>
<td>700</td>
<td>0.230</td>
<td>0.130</td>
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<tr>
<td>800</td>
<td>0.110</td>
<td>0.090</td>
</tr>
<tr>
<td>900</td>
<td>0.060</td>
<td>0.068</td>
</tr>
<tr>
<td>1000</td>
<td>0.040</td>
<td>0.045</td>
</tr>
<tr>
<td>1100</td>
<td>0.020</td>
<td>0.023</td>
</tr>
<tr>
<td>1200</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>
3.2.1. Fire design resistance for “I” section profile, without local buckling

According to Brazilian Standard, we have the following.

3.2.1.1. Tension

\[ N_{y,\theta,Rd} = \frac{A_g k_{y,\theta} f_y}{\gamma_{a,\theta}}, \]

where \( N_{y,\theta,Rd} \) is the design value of the tension resistance of the gross cross section, in fire, for a temperature \( \theta \), \( A_g \) the area of cross section, and \( k_{y,\theta} \) the yield strength reduction factor.

3.2.1.2. Compression (flexural buckling)

\[ N_{fi,\theta,Rd} = \frac{\rho_0 A_g k_{y,\theta} f_y}{(1 + \lambda_{0,\theta})\gamma_{a,\theta}} \quad \text{for} \ 0 \leq \lambda_{0,\theta} < 0.2, \]

\[ N_{fi,\theta,Rd} = \frac{\rho_0 A_g k_{y,\theta} f_y}{1.2\gamma_{a,\theta}} \quad \text{for} \ \lambda_{0,\theta} \geq 0.2, \]

\[ \rho_0 = \beta_0 - \sqrt{\beta^2 - \frac{1}{\lambda_{0,\theta}^2}}, \]

\[ \beta_0 = \frac{1}{2\lambda_{0,\theta}^2} \left[ 1 + \alpha \sqrt{\lambda_{0,\theta}^2 - 0.04} + \lambda_{0,\theta}^2 \right], \]

\[ \lambda_{0,\theta} = \frac{\ell_{fi}/r}{\sqrt{\pi^2 k_{E,\theta} E/k_{y,\theta} f_y}}, \]

where \( N_{fi,\theta,Rd} \) is the design value of the buckling resistance, in fire, for a temperature \( \theta \), \( \rho_0 \) the reduction factor for flexural buckling for a temperature \( \theta \), \( \lambda_{0,\theta} \) the slenderness parameter for a temperature \( \theta \), \( \ell_{fi} \) the buckling length, \( E \) the modulus of elasticity of steel, \( \alpha_0 = 0.384 \), the parameter of imperfection on fire, and \( r \) the radius of gyration.

3.2.1.3. Bending moment (lateral–torsional buckling)

\[ M_{fi,\theta,Rd} = \kappa_1 k_2 \frac{k_{y,\theta} M_{pl}}{\gamma_{a,\theta}} \quad \text{for} \ \lambda \leq \lambda_{p,\theta}, \]

\[ M_{fi,\theta,Rd} = \frac{\kappa_{y,\theta}}{1.2\gamma_{a,\theta}} \left[ M_{pl} - (M_{p/e} - M_{cr}) \frac{\lambda - \lambda_{p,\theta}}{\lambda_{\tau,\theta} - \lambda_{p,\theta}} \right] \quad \text{for} \ \lambda_{p,\theta} < \lambda < \lambda_{\tau,\theta}, \]

\[ M_{fi,\theta,Rd} = \frac{k_{E,\theta} M_{cr}}{1.2\gamma_{a,\theta}} \quad \text{for} \ \lambda > \lambda_{\tau,\theta}. \]
\(\lambda_{p,\theta}\) and \(\lambda_{r,\theta}\) are calculated as \(\lambda_p\) and \(\lambda_r\) with \(k_{y,\theta} f_y\) and \(k_E \theta E\) instead of \(f_y\) and \(E\), except in simple supported beams under concrete slab, where \(\lambda_{p,\theta} = \lambda_p\) and \(\lambda_{r,\theta} = \lambda_r\), where \(M_{fi,\theta,Rd}\) is the design value of the bending moment resistance of the cross section for a uniform temperature \(\theta\) and \(\kappa_1\) the adaptation factor for non-uniform temperature in a cross section. For a beam exposed on all four sides, \(\kappa_1 = 1.00\) and for a beam exposed on three sides, with a concrete slab on side four: \(\kappa_1 = 1.40\), \(\kappa_2\) the adaptation factor for non-uniform temperature along the beam. At the supports of a statically indeterminate beam: \(\kappa_2 = 1.15\) and in other cases: \(\kappa_2 = 1.00\), \(M_{p/} = Z f_y\) the plastic bending moment, \(Z\) the plastic section modulus, \(M_r = W (f_y - \sigma_r)\) the limiting buckling moment, \(W\) the elastic section modulus, \(\sigma_r\) the residual stress, \(\lambda_p = 1.75 \times \sqrt{E / f_y}\) the slenderness parameter corresponding to the \(M_{p/}\), \(\lambda_{p,\theta}\) the slenderness parameter for a temperature \(\theta\), corresponding to \(M_{p/}\), \(\lambda_r\) the slenderness parameter corresponding to \(M_r\), determined from \(M_{cr}(\lambda_r) = M_r\), \(\lambda_{r,\theta}\) the slenderness parameter for a temperature \(\theta\), corresponding to \(M_r\), and \(M_{cr}\) the elastic buckling moment.

3.2.1.4. Shear

\[
V_{fi,\theta,Rd} = \kappa_1 \kappa_2 \frac{k_{y,\theta} V_{Rk}}{\gamma_{a,fi}} \leq V_{Rk}.
\]

\(V_{Rk}\) is the characteristic value of shear resistance of the gross cross section for normal temperature, and \(V_{fi,\theta,Rd}\) the design value of shear resistance of the gross cross section, in fire, for a temperature \(\theta\).

3.2.1.5. Bending and axial compression

\[
\frac{N_{fi,Sd}}{N_{y,fi,\theta,Rd}} + \frac{M_{x,fi,Rd}}{M_{x,fi,\theta,Rd}} + \frac{M_{y,fi,Sd}}{M_{y,fi,\theta,Rd}} \leq 1.0
\]

and

\[
\frac{N_{fi,Sd}}{N_{fi,\theta,Rd}} + \frac{C_{mx} M_{x,fi,Sd}}{(1 - N_{fi,Sd} / N_{ex,fi,\theta}) M_{x,fi,\theta,Rd}} + \frac{C_{my} M_{y,fi,Sd}}{(1 - N_{fi,Sd} / N_{ey,fi,\theta}) M_{y,fi,\theta,Rd}} \leq 1.0,
\]

\[
N_{y,fi,\theta,Rd} = \frac{A_g k_{x,\theta} f_y}{\gamma_{a,fi}}, \quad N_{e,fi,\theta} = \frac{\pi^2 k_{E,\theta} EI}{\ell^4},
\]

where \(N_{fi,Sd}\) is the design value of the axial tension, \(M_{x,fi,Sd}\) the design value of the bending moment, in fire, about axis \(x-x\), \(M_{y,fi,Sd}\) the design value of the bending moment, in fire, about the axis \(y-y\), \(M_{x,fi,\theta,Rd}\) the design value of the bending moment resistance of the cross section, about axis \(x-x\), for a temperature \(\theta\), \(M_{y,fi,\theta,Rd}\) the design value of the bending moment resistance of the cross section, about axis \(y-y\), on fire, for a temperature \(\theta\), \(C_{mx}, C_{my}\) the coefficient applied to bending term in interaction formula (member subject to combined flexure and axial force), \(N_{e,fi,\theta}\) the value of elastic buckling load, and \(I\) the moment of inertia.
3.3. Example based on the application of the Brazilian Standard simplified method—critical temperature calculation

Based on Sections 3.1–3.3 and Table 3, it is possible to determine the critical temperature of each structural element. Two examples are presented as follows.

3.3.1. Columns without local buckling

From Section 3.2.1.2, we have

\[ N_{f_i,\theta,Rd} = \frac{\rho \, A_g \, k_{y,\theta} \, f_y}{1.2 \, \gamma_{a,fi}} \geq N_{f_i,Sd} \quad \text{for } \lambda_{\theta,0} \geq 0.2, \]

where \( N_{f_i,Sd} \) is the design value of tension in fire; hence,

\[ k_{y,\theta} \geq 1.2 \eta \left( \frac{\rho}{\rho_{\theta}} \right) \frac{\gamma_{a,fi}}{\gamma_a}, \]

where

\[ \eta = \frac{N_{f_i,Sd}}{N_{Rd}}, \]

\[ N_{Rd} = \frac{\rho A_g f_y}{\gamma_a}, \]

\[ \rho = \beta - \sqrt{\beta^2 - \frac{1}{\lambda_0^2}}, \]

\[ \beta = \frac{1}{2 \lambda_0^2} \left[ 1 + \alpha \sqrt{\lambda_0^2 - 0.04 + \lambda_0^2} \right], \]

\[ \lambda_0 = \frac{E}{\pi r \sqrt{f_y}} \]

\[ \frac{\gamma_{a,fi}}{\gamma_a} = 0.9 \quad \text{and} \quad \alpha = 0.384 \quad (\text{according to the Brazilian Standard}). \]

From Table 3, we have

for \( 500^\circ C \leq \theta \leq 600^\circ C \),

\[ k_{E,\theta} = 2.05 - 0.0029 \theta, \]

\[ k_{y,\theta} = 3.33 - 0.0031 \theta, \]

and for \( 600^\circ C \leq \theta \leq 700^\circ C \),

\[ k_{E,\theta} = 1.39 - 0.0018 \theta, \]

\[ k_{y,\theta} = 1.91 - 0.0024 \theta. \]

Finally, starting with the values of \( \lambda_0 \) and \( \eta \), it is possible to calculate \( \theta_{cr} \), as is illustrated in Figs. 1 and 2 [10].
3.3.2. Simply supported beams under slab, without local or lateral buckling

From Section 4.3.1.3, we have

\[ M_{f_{i,\theta,Rd}} = \frac{k_{y,\theta} f_y Z_x k_1 k_2}{\gamma_{a,fi}} \geq M_{f_{i,Sd}}, \]

hence

\[ k_{y,\theta} \geq \frac{\eta}{k_1 k_2 \gamma_{a,fi}} \gamma_{a}, \]

where

\[ \eta = \frac{M_{f_{i,Sd}}}{M_{Rd}}, \]

\[ k_1 = 1.4; \quad k_2 = 1.0, \]

\[ \frac{\gamma_{a,fi}}{\gamma_{a}} = 0.9. \]
From Table 3, we have for $600 \, ^\circ C \leq \theta \leq 700 \, ^\circ C$

\[ k_{E,\theta} = 1.39 - 0.0018\theta, \]
\[ k_{y,\theta} = 1.91 - 0.0024\theta. \]

Finally, starting with the value of \( \eta \), it is possible to calculate \( \theta_{cr} \), as is illustrated in Fig. 3 [10].

4. Conclusion

This work introduces to the international scientific community the recently released Brazilian Standards on fire steel structures design. These standards are based on international publications adapted to Brazilian structure standards. They are the first South American Standards on this subject and are considered a big advance in Brazil, having in mind their absence until now. An application of the Standards is also presented.

References


