Structural Fire Engineering of Building Assemblies and Frames

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Introduction

Use of the temperature-dependent thermophysical material properties, shape geometry, and fundamental heat transfer and structural principles, in combination with available fire test data, can enable several distinct levels of engineering/calculation methods of fire resistance. The simpler computational methods, such as those in ASCE/SFPE 29-05 [1], are semi empirically based on standard fire test results. They provide fire resistance ratings for members and assemblies that do not directly match listed assemblies to meet prescriptive code requirements. Higher-order fire simulations and structural analyses can be used as performance-based design alternatives to achieve a solution to overall fire safety.

Substantial fire-induced damage is expected after a severe (fully developed or post-flashover) fire exposure, not only to the building content and finish but also to the structural elements. It is not uncommon for well-designed, ductile, and properly functioning fire-resistive framing systems to experience visible distortions, cracking, permanent damage, and deflections that, in floors, can be on the order of 12–24 in. (300–600 mm), or more, without collapse.

In the following sections, several computational approaches to the determination of the fire resistance of building construction are summarized, independent of any requirements of a particular building code or design standard. These can be considered generally applicable to any structural material. The specific provisions of the governing building code and design standard(s) for a given project must be consulted for any engineering applications.

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Limit States Design

Design for structural fire resistance and safety (i.e., for the possible strength limit states, in general) to avoid collapse requires that the structural resistance be greater than the applied load effects. This strength limit can be symbolically expressed in Equation 52.1 as

$$R_{\mathrm{fire}} \geq L_{\mathrm{fire}} \tag{52.1}$$

where

R_{fire} = Available structural resistance under the particular high-temperature conditions, including the effects of degraded material properties

L_{fire} = Design values of the load effects (direct effects resulting from the applied loads and indirect effects resulting from restrained thermal expansion) expected to be simultaneously acting during the fire event

For critical facilities that need to continue operations immediately after fire events, it is also possible that such an engineering approach

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could be used to not only prevent collapse but to also minimize fire-induced damage. Such an objective could be accomplished through application of more stringent deflection controls in the design, to which conventionally rated fire resistive assemblies would typically not be subjected.

Because of the assumed accidental and extreme nature of this fire load and response condition, the safety/failure check should be conducted only in the ultimate limit state (ultimate strength or load and resistance factor design, LRFD) design realm, and not with the more restricted allowable or working stress design methods.

The simplest fire resistance calculations for individual structural members (beams and columns) and assemblies (walls and floor systems) are developed from best-fit, regression equations of ASTM E119 [2] fire test data and ratings. There are several shortcomings and limitations to standard fire tests and their derived fire resistance ratings, such as ASTM E119. Some are circumstantial as the cost of the tests, the time required to build specimens and do the tests and the limited number of facilities. Some others are more fundamental as the fact that only single elements can be tested as opposed to complete structures, the fact that the size of the tested elements is often smaller than the size or real elements, the fact that it is very difficult to control and have a precise idea of the boundary conditions in an experimental setup (perfect hinges and perfectly fixed supports are not easily realized in practice) and the variability inherent to experimental processes that make it nearly impossible to make controlled parametric analyses.

For a more complete assessment of structural load and response variables, Equation 52.1 provides the essential underlying criterion for strength adequacy under fire conditions, for both structural members and entire framing systems. More sophisticated analyses for members and frames rely on this basic limit state comparison of Equation 52.1 more explicitly. The degradation of the construction material properties at high temperatures; fire time-temperature values; type, thickness, and properties of the fire protection material(s); boundary conditions; connection response; and thermal strains are the key variables that should be included in this type of analysis.

Resistance

The two most important fire effects that alter a structure's resistance from that at ambient are the high-temperature degradation of its mechanical properties (strength and stiffness) and thermally induced strains. These softening, weakening, and damage to even noncombustible construction materials directly lead to a progressive reduction of load-carrying resistance at higher temperatures. Meanwhile, fire-induced thermal elongations can (1) lead to displacements so large that they influence the effects of action (a term used to designate bending moments, axial forces or shear forces) in the structure or, (2) when restrained, generate additional effects of action, typically in the form of compressive forces. These dual responses demonstrate that fire is clearly time dependent with effects on both the load and the resistance sides of Equation 52.1. Similar to the real time–history response of a structure subjected to an earthquake, load-resistance interactions exist that usually give rise to nonlinear structural behavior and permanent distortions/damage.

For example, a floor system may see the load bearing mode changing from bending at ambient temperatures to one with combined bending and axial compression; and, finally, during the large deflection and high-temperature stages of fire exposure, it may experience combined bending, axial tension and compression (catenary action in the beams and membrane action in the slab). This redistribution of the load-carrying capabilities of a typical floor system in a building from simple flexure under service conditions to catenary action at ultimate is schematically illustrated in Fig. 52.1.

The intent of the general design of Equation 52.1 is to verify the adequacy of structural

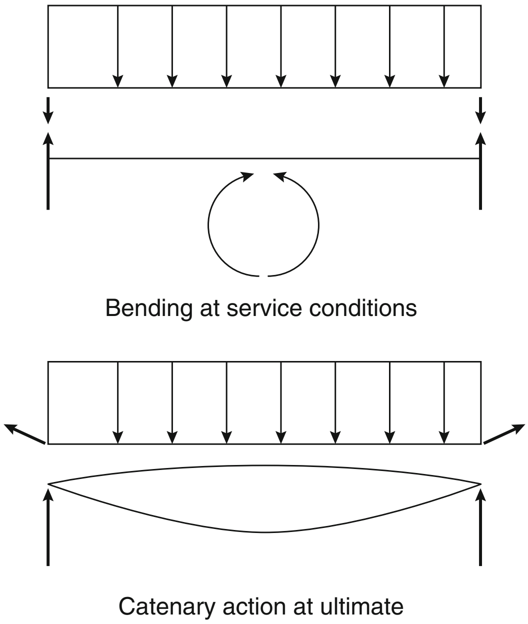


Fig. 52.1 Change in structural resistance of floor beams from bending to catenary action

resistance throughout the entire fire duration and response range or, alternatively, during a required period of performance time. The critical strength condition may not always occur at the maximum fire exposure time, especially with so-called natural design fires that have a burnout or cool-down stage. Member connections adjacent to the fire must also accordingly accommodate the forces, moments, and distortions generated by the event, with adequate ductility to avoid any failure(s); the same holds for all remote elements or parts of the structure that are not directly affected by the fire (in the sense that their temperature remains unchanged). An alternative and generally equivalent limit state formulation to that in the strength domain can be established in the time domain. Use of conventionally pre-established critical member temperatures alone does not provide a totally meaningful or comprehensive solution under fire exposure, because this is only one aspect of the structural response. It does not explicitly address whether the member will fail to support its load demands under such conditions.

Reliability

The general limit states formulation of Equation 52.1 is implicitly based on probability theory and a low, but societally acceptable, failure risk. Such a de minimis risk, or the threshold level below which an event is not of regulatory concern, is a probability of failure of approximately 10^{-6} or less per year [3]. The reliability index to sufficiently control this probability of occurrence is at least about 3.8, which could be used as the target design reliability baseline if one were not using a recognized fire design standard or code. The design reliability includes both the extreme load effects and the lower-bound expected resistance at ultimate strength, as well as the probability of occurrence of an uncontrolled, fully developed fire. The available statistics of the selected or specified load combination, design fire, and structural framing enable a rational calculation of the particular combined reliability of any such design scenario, at least in an approximate sense.

Similar to the empirical fire resistance ratings of building construction elements, fire engineering design of a compartment space is intended to control its vulnerability to localized structural member or assembly failures for a design fire exposure. The potential for fire movement beyond the compartment of origin to other areas or floors can also be duly considered and rationally evaluated, as well as effects of simultaneously occurring fires in multiple compartments and/or floors, relative to potential development of progressive (or disproportional) catastrophic failure of the framing system. For these conditions, a much more complex interaction of thermal and structural action over the affected framing takes place, with significantly greater demands placed on modeling capabilities, computational power, and convergence time for concurrent time–history solutions. The need for suitably customized finite element software, project budget allowances, and computational resources lead to the situation that these types of applications are not usually performed for smaller projects. This more advanced fire

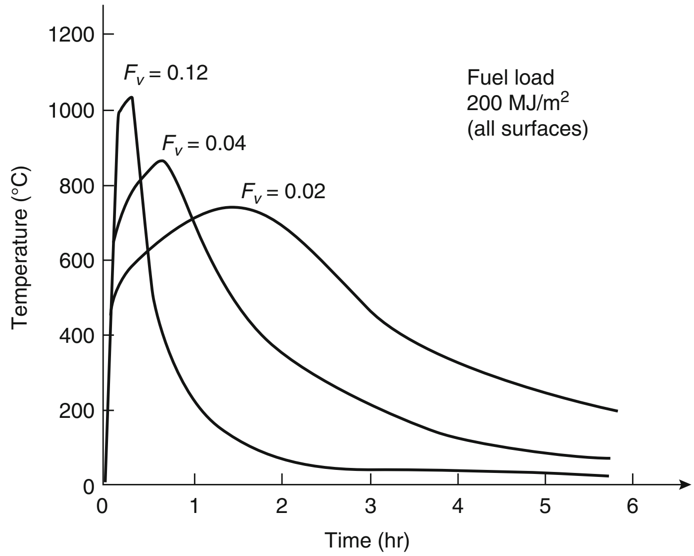


Fig. 52.2 Real time–temperature curves for different ventilation factors, F_v, with a constant fuel load [7]

engineering is essentially applied to higher importance, higher risk, high-rise and/or otherwise unique landmark buildings.

Fire Exposures

This section provides a short introduction to the information covered much more in depth in Chaps. 29, 30, 53, 54, and 55. ASTM E119, *Standard Test Method for Fire Tests of Building Construction and Materials* [2], has long provided (since 1918) the basis for the test fire used in establishing fire ratings of structural building elements in the United States. UL 263, *Fire Tests of Building Construction and Materials* [4], and NFPA 251, *Standard Methods of Tests of Fire Resistance of Building Construction and Materials* [5], are similar documents, as is the international standard ISO 834 [6]. All these standard fire exposure curves can be considered essentially identical. This ASTM E119 standard fire is fast starting, hot, and rising with an equivalent burning temperature of 1000 °F (538 °C) after only 5 min.

An uncontrolled natural fire has distinct stages of growth, fully developed (flashover) burning, and finally decay. In contrast, the ASTM E119 standard fire has no decay or burnout branch but specifies ever-increasing furnace compartment temperatures with time that can reach 2300 °F (1260 °C) at 8 h, if testing were to reach this duration. Normally, ASTM E119 fire tests for listing of building construction elements are not conducted for more than 3–4 h.

A natural fire eventually consumes its combustibles in a finite time that is dependent on the initial quantity and type of fire load and the amount of ventilation in the compartment. Hence, the actual fire load and ventilation present in the room will determine the nature, intensity, and duration of a real fire. Uncontrolled, well-ventilated fires reach higher temperatures than poorly ventilated fires, but they burn faster and have a shorter duration for the same fuel. This is illustrated in Fig. 52.2 [7], in which F_v is a ventilation factor.

In Fig. 52.3, the ASTM E119 standard time-temperature curve is superimposed on several representative real fire curves for various fuel loads and a constant ventilation factor. The maximum fire temperature, its decay phase, and its fire time duration are significantly affected by the fuel content and ventilation, and are quite different from the standard time-temperature curve. As expected, higher fuel loads cause longer and hotter fires under uncontrolled conditions. The standard fire time-temperature curve between

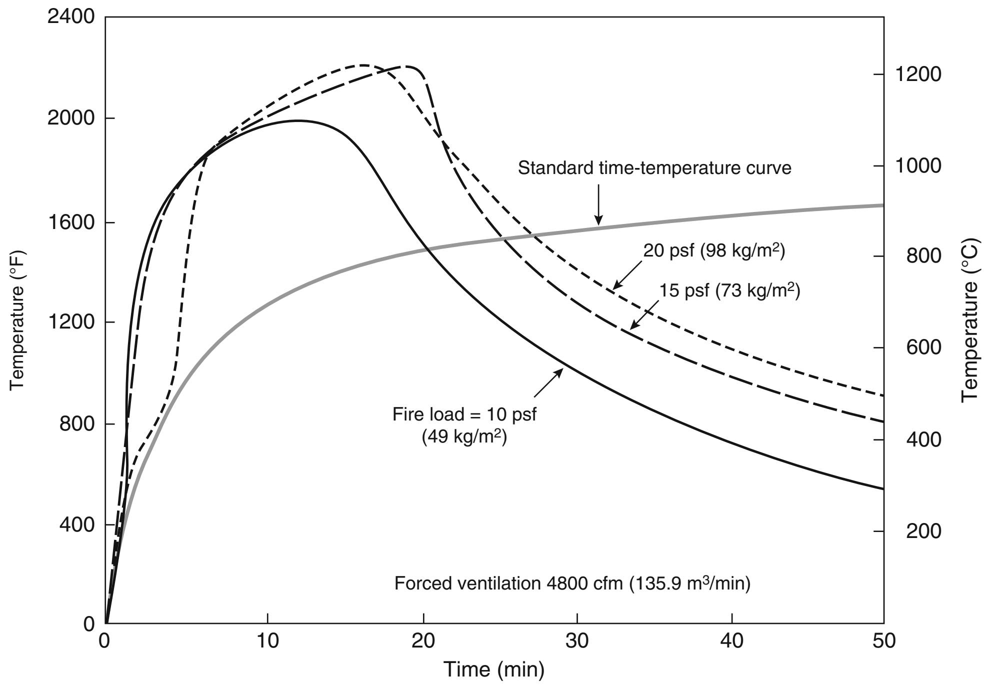


Fig. 52.3 ASTM E119 standard fire and real fire time-temperature curves

1 and 4 h provides a good order of magnitude of the room temperatures that will be encountered in a real conventional fire in many cases. However, in real uncontrolled fires, these high temperatures most likely occur over only a relatively short time interval.

For faster starting and hotter fires, such as those that occur from petrochemicals or other hazardous materials, a standard fire exposure more severe than that given in ASTM E119 may be more appropriate. ASTM E1529 (UL 1709), *Standard Test Method for Determining Effects of Large Hydrocarbon Pool Fires on Structural Members and Assemblies*, defines such a standard fire exposure that reaches and remains at about 2000 °F (1100 °C) after 5 min.

Mathematical representations of the ASTM E119 [2], other standard fire time-temperature curves, and a variety of real compartment fire time-temperature formulations can be made for analysis and design purposes. Various such fire models and parametric curves can be found in the literature, such as the *SFPE S.01 2011 Engineering Standard* [8] and Eurocode 1 [9]. Fire models usually can predict a fire temperature-time history only within a single compartment.

The simplest fire models predict the resulting evolution of the gas temperature within the fire compartment. This is the case for parametric fire models or one zone models. This gas temperature is unique at any time during the fire, which is mostly relevant for a post-flashover situation. More refined models can provide a more detailed picture of the situation in the fire compartment, which is needed if the pre-flashover phase has to be described or in the case of localized fires, such as an axisymmetric fire plume. Two zone models consider a vertical separation between a higher zone containing the hot combustion gases and a lower zone containing cold uncontaminated air. More recent travelling fire models represent the fact that, even in a post-flashover fire, the maximum combustion and maximum gas temperature do not occur simultaneously in the whole compartment [10]. Computational Fluid Dynamics

(CFD) models can also be used to give a very detailed description of the situation in terms of gas temperature, pressure, velocity, in millions of cells of the compartment.

Interior fire extensions through window openings presents another scenario category which analytical modeling can also simulate. Eurocode 1 [9] presents a standard formulation to represent such events.

In performing a structural fire resistance analysis and design, either the well-established standard fire, such as E119, or a natural design fire for the particular building and occupancy can be employed to determine both its R_{fire} and L_{fire} effects. For selection of a natural design fire, surveyed or code-specified fuel loads are an essential demand input. The fuel load can be separated into fixed and variable classifications. The fixed combustibles are those that remain essentially unchanged within the bounded compartment (i.e., interior wall, floor, and ceiling finish, structural framing), whereas the variable fuels are those combustibles that can change over time based on occupancy (furnishings and contents). The fixed fuel load should be determined from a project-specific survey or estimated for noncombustible construction. This fixed fuel load is added to the variable quantity to comprise the total fire design load. Typically, in the absence of a project-specific survey, the variable fuel load could be represented as a nominal design level for the fire demand to provide for suitable overall reliability. Adjustments to higher percentile fuel levels may be specified for selected building conditions, such as lack of automatic suppression/sprinkler systems, larger compartment sizes, occupancy, height/criticality of building, and the risk that the design fire could propagate beyond the compartment of origin to adjacent areas or floors.

As a guide for some typical variable fuel contents expressed in terms of potential heat energy release per unit floor area, Eurocode 1 [9] can provide a reasonable initial reference for common occupancies such as residential, office, commercial, and library. The expected statistical distribution parameters (mean, standard deviation) of the fuel loads are provided as well as values for certain non-exceedance percentiles. Older publications, and some current work, also cite the potential fuel in terms of a weight density per floor area, pounds per square foot (psf), or the like for equivalent amounts of wood combustibles. NFPA 557 [11] is a comparable U.S. fire load standard based on review of the literature and fuel data surveys. The particular design value for variable fire load density depends on the selected or required percentile of non-exceedance, which will vary depending on the height and criticality of the building project, presence of other fire safety measures (including active fire suppression systems), and areas of compartments. Special fuel content surveys may also be undertaken to assess this load density for particular buildings. Hence, selection of the appropriate design fire load, in the absence of a building code requirement, is subject to the responsible professional's judgment and approval by the authority having jurisdiction. For more information, refer to Chap. 35.

High hazard occupancies must be individually determined for each given project.

Overview of Heat Transfer Analysis

The structural behavior of a structure subjected to fire is directly dependent on the temperatures that are induced in this structure by the fire. The determination of the fire resistance of a structure, or of a member, by calculation thus starts with the determination of the temperatures in the structure or in the member. Once a design fire has been selected, a heat transfer analysis is thereby the next step to determine the resulting temperatures in the structure. More discussion of heat transfer is contained in Chap. 34.

Usually, lumped mass single temperature, one-dimensional or two-dimensional analyses will suffice. Most engineered structures are indeed based on linear or planar members, such as beams or columns, frames, floors or walls. If the boundary conditions imposed by the fire to such members are the same along the length of a linear element or on the surface of a planar element, the temperature does not vary along the length or in the plane of the element. The determination of the temperature distribution is thus usually reduced to a two-dimensional problem on the section of linear elements or to a uniaxial (one-dimensional) problem across the thickness of a planar element (at least for walls and floors that have a constant thickness and are of a single material). Some slabs, like hollow core concrete slabs or composite steel-concrete floors with trapezoidal corrugated steel sheets, require a two-dimensional analysis. The problem in that case is similar to the two-dimensional problem that has to be solved for the beam sections.

Three-dimensional thermal analyses may be performed locally, for example in complex joints between linear elements such as, e.g. beam-column joints. Some walls can in fact be made of concrete blocks with internal cavities or masonry bricks that, together with the mortar joints, form a three-dimensional structure. However, thermal analyses considering the walls with such a level of details are seldom performed.

The size, nature, and type of fire protection (insulation) used for the structural member(s) are essential factors in how quickly, and to what level, the structural material's temperature will rise when subjected to the effects of a fire. Unprotected noncombustible members will heat up most rapidly to eventually reach thermal equilibrium with the fire gas temperature, whereas combustible elements will eventually ignite and be consumed as a fuel.

The heat transfer computation will determine a temperature profile for the higher-order analysis option, within a given structural member at a given fire duration. The one-dimensional heat transfer solution can produce temperature gradients in one direction, commonly through the material thickness. The further simplified, lumped mass analysis is limited to just solution of a single material temperature, hence assumed uniform member temperature. A general flowchart schematically showing the steps and options for this fire and thermal part of the engineering approach is given in Fig. 52.4. Usually, the single fire compartment scenario would govern, as assumed in the standard fire resistance tests and ratings. This scenario is repeated with the fire starting in each possible compartment and/or floor.

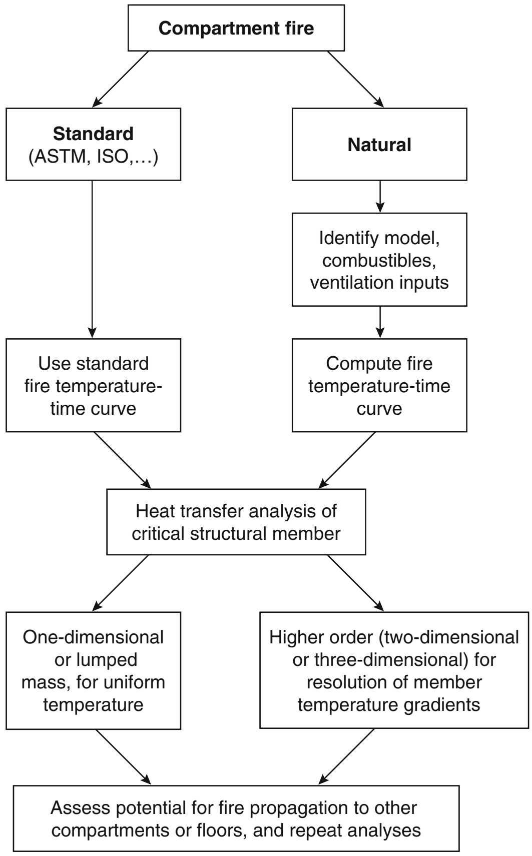


Fig. 52.4 Schematic of fire engineering process to determine maximum temperatures in individual structural members

Most often, a weak coupling is considered between the thermal and the structural analyses. This means that the effects of the temperature variation in the structure on the behavior of the structure are taken into account, whereas the effects of the structural behavior on the temperature development are neglected. One exception could be the failure of partition walls or floors that extend the fire zone.

The longitudinal heat flux that may exist along linear members or in the plane of planar members, the heat flux that may exist from members to members, and the temperature distribution in the joints are briefly discussed in the following section.

Variation of Temperatures along the Members The world in which we live is geometrically three-dimensional. The temperature distribution in construction members is thus three-dimensional in general. Even if the hypothesis of a two-dimensional temperature distribution in linear members is common and practical, longitudinal heat fluxes and temperature gradients will indeed be present in practical situations, essentially in the vicinity of the joints that connect the members and especially in the situation where some members are subjected to the heating of the fire, while some others connected to the same joint are in an adjacent compartment where the cooler ambient situation still prevails.

It is possible to determine the longitudinal heat fluxes, the temperature gradients along the length of the members, as well as the influence of these temperature variations on the mechanical behavior of the members or of the complete structure. Very often, such analyses have been performed for the idealized and somewhat academic situation of prismatic members that have, for example, one part of the length subjected to the fire and the rest surrounded by air at ambient temperature. But these members have, in most studies, the same section along their length and are themselves isolated from the rest of the building.

In reality, each joint is in general a geometrically complex object that extends in all directions (for example, two vertical elements, two horizontal elements spanning in the main direction, and two horizontal elements in the secondary direction). The joint connects members of different types and different sections. It may also be thermally influenced by the presence of nonstructural elements, such as suspended ceilings or vertical walls. Very often, the presence of stiffeners, bolts, and various connecting elements that all have an influence on the heat transfers through the joint will further complicate the situation. This explains why the determination of the true three-dimensional temperature distribution for all the joints in a real building would be an extremely long and expensive process. Performing parametric analyses on various joint typologies and deriving practical design guides about the temperature distribution to be used in everyday design is probably beyond reasonable expectations.

In fact, the information contained in the complex three-dimensional temperature distribution that could eventually be determined in a particular situation could hardly be exploited in the analysis of the adjacent elements. This is because the three-dimensional temperature distribution would be determined on the basis of three-dimensional solid elements, whereas the structural members are usually modeled by oriented elements, either linear (beams and columns) or planar (floors and walls). It would thus be challenging to map the temperature distribution determined in a 3D object into oriented 2D members.

The simplified hypothesis of a temperature distribution in the members that is uniform along the length per segment and that varies abruptly only at the joints is thus normally used. It is somehow justified by the observations made during the analysis of ideal prismatic members. Even if the thermal environment varies abruptly along the length of a prismatic member, the affected zone in terms of temperature variation is quite restricted in length [12]. There is an observable zone in the cold compartment where the temperature in the member is increased, but this temperature increase is smaller than in the adjacent heated part, and failure is not likely to appear here. There is also a short zone in the exposed part where the temperatures are reduced, but it is assumed to be on the safe side to neglect the effects of this colder zone. Typically, the affected zone is twice as long in the cold compartment as in the hot compartment.

The temperature in the joint is of crucial importance if the mechanical behavior of the joint has to be determined. For joints between steel members, the hypothesis is often made that the temperature in the joint is uniform and the mechanical behavior is determined as a function of this temperature. This temperature can, for example, be determined on the basis of the thermal massivity of the joint as a three-dimensional object, i.e. the ratio between the exposed surface of the joint and the volume of steel. If a slightly more refined thermal model is required, it is possible to calculate the temperature in each component with its own massivity, but this amounts to neglecting the heat flux between the components, which is certainly not true in a thermally composite joint. Numerical tools can be used to derive a very detailed temperature distribution in the joint. To the authors' knowledge, joints have so far been analyzed on the hypothesis that the joint is fully exposed to the fire on all sides. The analysis of joints that are only partly subjected to the fire has yet to be done.

Thermophysical Properties of Fire Protection Materials The variation with high temperatures of key input properties such as conductivity, specific heat, and density for the various spray-on and intumescent coating products, gypsum boards, and the likes will directly affect the analytical predictions of temperatures in the structural members. There are relatively little published data on these, and they are often rather approximate, incomplete, or conflicting. A parametric sensitivity analysis would quantify the range of possible results corresponding to the input variations. Moreover, the adhesion/cohesion of these materials to the substrate and/or the integrity of their protective envelope during the fire are important response characteristics that cannot ordinarily be evaluated solely from analytical models. Experimental evidence of the materials' suitable fire protection performance in this regard is necessary to enable the analytical assumptions that the materials remain in place, as installed, throughout the exposure duration.

The relevant material properties at high temperatures are discussed in Chap. 9 in this handbook.

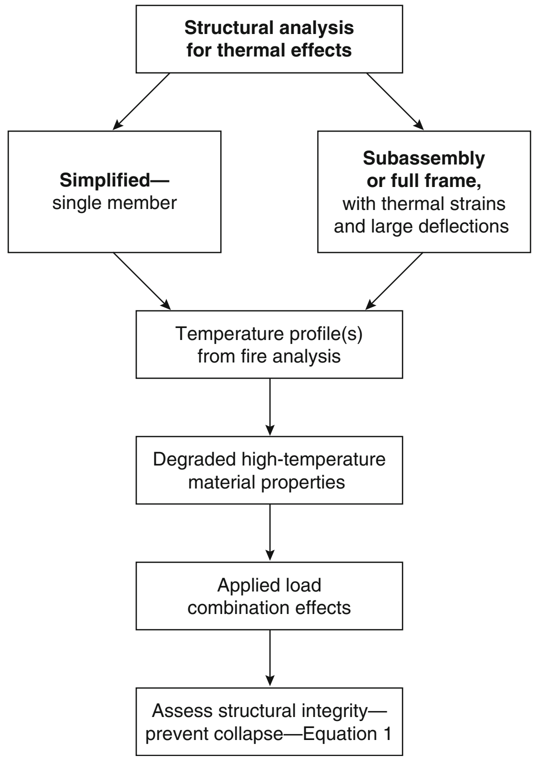


Fig. 52.5 Schematic of structural analysis for fire effects

Overview of Structural Analysis

Figure 52.5 outlines the subsequent structural analysis for loading effects that is performed once the fire-induced temperature(s) in the structural member or system have been determined. A simplified single member-by-member or a subassembly/frame structural analysis can be conducted. The fire-induced thermal expansion and structural restraints are unique features of structural-fire interaction. For the single member assumption, the member boundary conditions and load effects are taken only from the structural analysis at ambient temperatures, thereby neglecting any thermal strain effects on the selected member or the surrounding structure. A subassembly or full-frame structural analysis that includes the thermal response of the fire-exposed member(s) will offer the most complete representation of the response, including any restraining effects on or by the adjacent framing due to the fire, which would otherwise not become manifest in the single member method. More information on structural analysis is contained in Chaps. 53–55.

Given the complex and advanced nature of the nonlinear structural response under high-temperature exposures, several unique factors that are not routinely considered for most design practice will need to be duly evaluated for analytical solution accuracy.

Local Member or Frame Instability These destabilizing effects due to slenderness of members in compression can be represented by the addition of the geometric stiffness matrix—or comparable idealizations—that accounts for the coupling of axial compressive forces on flexural stiffness. This second-order (or higher) effect can also be described as ensuring the equilibrium of the applied loads on the displaced structure, not its original geometry. The role of structural imperfections for compressively loaded elements may be important in this regard. Initial member crookedness, expected geometrical misalignments, and/or construction tolerances in the building framing or connections may need to be further assessed and studied. These initial imperfections would be reflected as input data to the model(s), in lieu of the perfectly straight original geometry. Larger compressive loads will effectively soften the member or element, which consequently leads to amplified lateral/transverse deformations and bending moments relative to linear elastic assumptions. This structural weakening can eventually progress to the final buckling limit state, when the element becomes unstable and is considered to have failed, at large distortions and with little, if any, residual stiffness. This instability is analytically signaled by the stiffness matrix becoming singular, large displacements, and/or the occurrence of non-convergent numerical solution algorithms. Local buckling instabilities can be analytically modeled and detected only if the structural member is adequately discretized into appropriate beam, plate, brick, or shell elements. If each member is discretized by one single element, only overall member and frame instabilities can be detected in a nonlinear formulation. Linear elastic solutions cannot detect any instability.

Floor Slab Effects The structural effects of the concrete and deck floor slabs on the strength and stiffness of the steel beams and girders, for bracing and composite design with shear studs, must be properly modeled; the steel beams and the concrete slab work together in a composite action.

Thermal Strains The thermal expansion, and structural restraints, must be properly included, since this is a unique feature of structure-fire interaction, as discussed later.

Tensile Membrane Action of Composite Floors [13] Composite floor systems based on profiled steel sheets (decks) are designed for the normal situation to span in one direction, the direction of the ribs. The length of the span is in the order of magnitude of 10 ft, depending on the depth and thickness of the steel deck and on the eventual presence of shoring during casting of the fresh concrete. The traditional approach for ensuring an appropriate fire resistance to such systems was, until the end of the twentieth century, based on additional steel reinforcing bars located in the ribs in order to carry the tension force when the temperature in the steel profiles has increased and their load bearing capacity has vanished. Experimental tests [14, 15] and observations in real fires such as the Broadgate fire in London in 1990 or the Churchill Plaza in Basingstoke in 1991 have shown that a different load transfer mode may develop in the fire situation. Where the load transfer capacity by bending in the slab has been lost, the slab deflects and tension develops in the central part of rectangular slab panels. These tensile forces are supported by the steel reinforcing mesh that must be present in the concrete slab and are equilibrated by a compression ring that forms in the external parts of the slab. This allows leaving some of the infill beams that support the floor unprotected on the condition that the edge beams that form the new

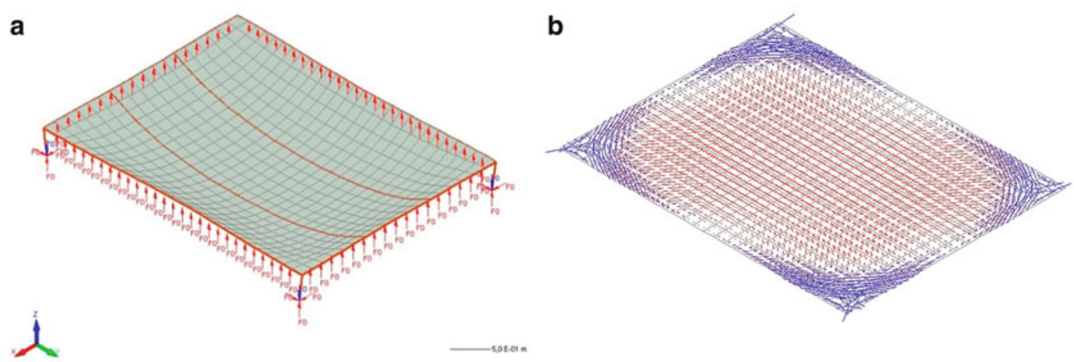


Fig. 52.6 Tensile membrane action. (**a**) Deformed shape of the floor. (**b**) Membrane forces in the floor

slab panel are able to carry the vertical reaction of the system. Figure 52.6 shows: (a) the deformed shape of the new slab panel of increased dimensions that forms during the fire when the two infill steel beams—represented by the **bold** lines—are left unprotected and (b) the distribution of membrane forces in this panel with compression near the edges and tension in the central part of the slab panel.

This mechanism is now well understood and is routinely used for designing composite steel-concrete floor systems, especially in the U.K.

Connection Moment-Rotation Behavior Building framing connections for ordinary design are usually idealized as being either fully rigid-FR (full bending moment transfer and maintain original angles between members) or simple (no bending moment capability and free rotation between members). Depending on the nature of the postulated collapse mechanism and its critical subassembly, it may become necessary to more rigorously model some of these connections with characteristic moment-rotation curves in place of the original simplifying design assumptions, in order to estimate the real joint flexibility and partial rotational restraints. Significant progress has been made in the last decade for understanding and modeling the stiffness and strength characteristics of connections in the fire situation [16]. It has been shown that not only the strength but also the ductility of the joints is essential for ensuring a satisfactory behavior, especially during the cooling phase of a fire [17].

Nonuniform Heating A temperature gradient through the member depth/thickness causes differential thermal strains between the hotter and cooler external surfaces. A temperature gradient will exist, for example, in steel beams with concrete floors under fire exposures, because the floor slab keeps the top of the steel beam cooler. Perimeter columns and truss members will also likely experience some degree of non-uniform heating in a real fire. These thermal effects will depend on whether it is assumed that the fire totally engulfs a given structural member. If so, a similarly uniform heating exposure on all sides can be expected with no temperature gradient or bowing, such as for an interior column. In simply supported members, these thermal gradients give rise to the so-called "thermal bowing." This thermal bowing/curvature will usually be toward the hotter side of the member. These induced thermal curvatures reduce the load-carrying capability of the members in compression due to P-delta effects and, hence, may influence the stability of the columns and truss. Under restrained end conditions, these displacements cannot develop and the effects of actions are modified even at first order (no need of large displacements). A beam that has both ends fixed in rotation and is subjected to a thermal gradient on its depth will experience no bowing in any direction but the bending moment diagram will be changed; if the lower part of the section is hotter than the upper part, negative moments, or hogging, will develop on the whole length of the beam.

Material Strength Limit States Strength (due to ductile yielding, crushing, or tensile rupture) will govern the response of the common construction materials such as steel, concrete, masonry and wood. Yielding, rupture, or stability will often control the ultimate strength of steel members; heavier concrete and masonry have negligible tensile resistance and rely essentially on compressive strength, with steel reinforcement providing the primary tensile capacity and supplemental shear resistance. The extent and type of reinforcing details in concrete and masonry will greatly influence the ductile or brittle nature of subsequent behavior. Plain/unreinforced concrete and masonry will generally be quite brittle and much more susceptible to early cracking failures, whereas heavily reinforced and well-detailed members can perform in a ductile manner at large deformations. Yielding marks the major departure of the structural element's behavior from linear elastic to nonlinear. This nonlinearity is manifest by reduced material stiffness below Young's elastic modulus and by strength that is governed by the constitutive properties of that particular material. Available ambient and temperature-dependent stress-strain relationships for these materials can be used in this regard. Wood structures, with loss-of-section by charring effects, can be reasonably modeled through otherwise linear elastic assumptions. Conversely, steel and concrete structures usually require a full nonlinear analysis. Cracking, potential explosive spalling, and loss of concrete cover to the interior steel reinforcement will affect the reinforced or prestressed concrete member's integrity under fire, but this response is usually well beyond the capabilities of most software.

Collapse Prevention

The primary life safety objective of structural fire resistance is either to avoid collapse during the standard, or design, fire or delay collapse, in a real fire scenario, to a time when all occupants have safely evacuated the building. Collapse can be broadly classified as either local or global. Local collapse is failure of a single member, connection, or limited frame subassembly, whereas global, progressive, or disproportionate collapse produces a major cascading series of related failures triggered by the original local collapse. The latter is the much more dangerous and destructive in terms of both public safety and property loss.

With use of the single member, subassembly, or full-frame structural/fire analyses described earlier, one can determine the source and type of any initial failure. Ordinarily, identification or avoidance of this first structural failure will suffice for compliance with the basic safety requirements of the building codes.

However, more recently with concerns after several terrorist attacks, there is an increased awareness of the risks of disproportionate collapse, particularly for taller or monumental/historical buildings. The analytical determination of whether an initial local/member failure can propagate to further global or disproportionate structural instabilities may be difficult, or impossible, for most finite element software operating in a static analysis mode. Numerical solution convergence for any subsequent catastrophic and complex global collapse mechanisms can be much more easily achieved if the nonlinear software processes the analysis (with resulting large deformations, member failures, singularities, etc.) as a dynamic, rather than static, equilibrium problem, with an appropriate time step [18].

In addition, it may be necessary to conduct such simulations in full three-dimensional space, with adequate and appropriate discretization of the potentially affected members, that is, many more model nodes and elements, to reach the best response fidelity. Such intricate simulations are likely to require extensive computing resources, time, and effort to accomplish.

Structural Load Combinations for Fire Resistance

The applied load effects, L_{fire}, for use in Equation 52.1 need to be determined from the loads and load combinations required by the applicable building code, and these will constitute the so-called mechanical, or nonthermal, effects on the heated structure. The common building design loads are dead (D), floor live (L), roof live (L_r), roof snow (S), rain (R), wind (W), and earthquake (E), several of which may or may not act simultaneously. Maximum structural stresses or deformations, therefore, will result from the critical combination of the loads. Building codes specify various combinations that must be checked. The most critical load combination may occur when one or more of the loads are not acting. In some codes or standards, provisions for design to withstand an extraordinary, extreme, or accidental event (A_k) may also be given, such as for fire, explosion, and vehicular collisions with building.

For the fire engineering problem, the design values of the loads are based on probabilistic considerations: the probability of failure from the effect of a fire (which must meet a certain target value) is the product of the probability of having a severe uncontrolled post-flashover fire by the probability of failure when this fire occurs. As the first of these two probabilities is much lower than 1, the probability of the second event, the one that is assumed when the fire resistance is calculated, can be higher than the target value. The design value of the load will thus be less in the fire situation than the full design live load that is normally specified for ambient temperatures. In addition to the normal (ambient) design load requirements, ASCE/SEI 7-10 [19] in the United States specifies use of the following extraordinary event gravity load combination for fire design:

$$1.2D + A_k + (0.5L \text{ or } 0.2S) \qquad (52.2)$$

where A_k symbolically represents the fire effects or typically the construction material's strength and stiffness reductions caused by the fire's heating.

This load combination in Equation 52.2 for extreme exposures is intended exclusively for application in limit states design with Equation 52.1.

Other international codes and standards specify comparable reduced load combinations to be used in combination with fire exposures. This contrasts with the full gravity design live load that is used in most standard fire resistance tests, such as ASTM E119. It should be recognized that the frequency of major building fires is relatively low due to the small probability of ignition coupled with flashover, due to occupants or fire department intervention and/or automatic fire suppression system's extinguishment of the fire before it becomes fully developed [3]. However, if flashover occurs, the uncertainties of the actual fire intensity, duration, spread, and localized heat distribution effects to the affected structural members (L_{fire}) are large relative to the variability of the structural fire resistance (R_{fire}). The coefficient of variation (COV) of the fuel load contents that serve as the fire combustibles is considered to be on the order of 0.50 or more, similar to common live gravity and environmental (wind, snow, earthquake) loads [3, 20], whereas the typical structural resistance COV is in the range of 0.10–0.20.

High-Temperature Effects on Structure

The two primary high-temperature effects on structural materials that are not exhibited under typical ambient conditions are thermally induced strains and degradation of the materials' mechanical properties (strength and stiffness). Because of these two effects, deflections of structural members during the longer duration, hot fires (post-flashover) can reach many inches or even several feet. This is at least an order of magnitude greater than the small elastic deflections, usually no more than about 1–2 in. (25–50 mm) that are normally contemplated for design service. These effects are only briefly described in the following sections, as there are existing sources for this detailed information, including other chapters in this handbook. The commonly used structural fire protection materials and systems are generically addressed in the final section.

Table 52.1 Coefficients of thermal expansion for steel and concrete at high temperature [21]

Structural and reinforcing steel	7.8×10^{-6}/°F (1.4×10^{-5}/°C)
Normal-weight concrete (NWC)	1.0×10^{-5}/°F (1.8×10^{-5}/°C)
Lightweight concrete (LWC)	4.4×10^{-5}/°F (7.9×10^{-6}/°C)

Thermal Strains

Strain is defined as change in member length divided by the initial length. In general, the total strain, ε_{total}, in a structural element, can be considered to be primarily composed of mechanical and thermal parts, as given in Equation 52.3.

$$\varepsilon_{total} = \varepsilon_{mechanical} + \varepsilon_{thermal} \tag{52.3}$$

Mechanical strain is related to applied loads and stresses, whereas thermal strain results only from the material's expansion or contraction due to temperature changes. Under fire exposures, the thermal strain is elongation in proportion to the material's coefficient of thermal expansion. For constant temperatures, the total strain is only the mechanical strain, because the thermal strain is zero. However, at elevated temperatures, high mechanical strains can develop even under negligible superimposed and dead loads due to restraint of thermal expansion.

If the member is fully restrained so that no total strain occurs, all the thermal expansion effects are converted into an equal and opposite mechanical strain (compression). These mechanical strains induce internal reaction forces and moments in the structure, which may lead to either material ultimate strength or strain becoming the governing limit state. Similar to seismic design, the ultimate strain, or deformation, rather than strength may become the failure limit for the more brittle construction materials that do not have sufficient ductility. For the opposite extreme, in the case of a simply connected but effectively thermally unrestrained member, all the thermal expansion will freely occur as part of the total strain and will not directly induce any additional mechanical strain (additional mechanical strain can occur because of second-order effects linked to the large displacements that the free thermal expansion produces). Therefore, for totally unconstrained thermal expansion and unloaded elements, total strain equals the thermal strain, with thermally induced mechanical strain and stress being zero.

The real boundary conditions for thermal restraint in buildings lie between these extremes and require a more in-depth analysis for an accurate determination. For such indeterminate framing, this deformation compatibility and force/moment equilibrium analysis under heating and applied loads is the key step in assessing the structural integrity of a member or subassembly.

AISC [21] provides the following thermal elongation strains (coefficients of thermal expansion) in Table 52.1 for material temperatures above 150 °F (65 °C), which are similar to the values given in other major international standards.

Constant coefficients of expansion lead to a thermal expansion that is proportional to temperature. This simplification closely approximates more detailed representations that can be found in the literature, and it is usually both practical and sufficient for design applications of simple elements. The thermal elongation strain resulting from a temperature increase of 500 °C in the material is about 0.004 for lightweight concrete (LWC) and 0.007–0.009 for steel and normal-weight concrete (NWC), which represents an increase of about 0.5 in./10 ft (4 mm/m) for LWC or about 1.0 in./10 ft (8 mm/m) for steel or NWC. These possible levels of elongation during a severe fire are clearly significant.

Thermal Degradation of Construction Material Properties

Combustibility is one broad, and important, fire classification of building materials. Noncombustible materials will degrade under the higher temperatures of a fire but will not burn. Combustible materials will not only degrade at higher temperatures but also ignite and burn, thereby adding to the fuel contents during a fire.

Building materials serve in the primary, load-bearing elements that are necessary to preserve the structural safety of the building in preventing partial or total collapse. The traditional building materials have been steel, concrete, masonry, and wood. Wood is the only combustible material of these four. In all cases, visible damage/distortions and degradation (potentially including cracking, dehydration, loss of section, charring, etc.) of the mechanical properties of all building materials occur under prolonged elevated temperatures.

Application of more advanced fire resistance solutions will require an explicit representation of the basic thermal and mechanical material properties at elevated temperatures, such as yield and ultimate stress, modulus of elasticity, coefficient of thermal expansion, thermal conductivity, and specific heat. The detailed material response and property variations at high temperatures of fire on concrete, masonry, and wood materials may be readily obtained from Chaps. 54 and 55 in this handbook, as well as from the published literature.

Structural Analysis

For each combination of loads that could be applied to the structure in the fire situation, the effects of actions, namely bending moments, axial and shear forces, and also the support reactions, have to be determined in the structure. This determination is the structural analysis.

Because of the indirect effects of actions, that is, those variations in the effects of actions caused by restrained thermal expansion, it may be necessary to perform the structural analysis continuously during the duration of the fire course. This has then to be made not only for every combination of loads, but also for any fire scenario that is being considered. The fact that large displacements created by unrestrained thermal expansion and the softening of materials vary constantly under increasing temperatures is another reason that may lead to the necessity to perform a structural analysis at every stage of the fire.

There are yet some simplifications that can lead to a less demanding process for the structural analysis. The conditions of these simplifications and the limitations of these simpler procedures are discussed in this section.

Structural Analysis Before the Fire

As a starting point, the effects of actions have to be determined in the structure under the design load combination in case of fire for the moment when the fire starts, a moment that is usually called $t = 0$, or time at zero.

In fact, some simplified methods used to determine the fire resistance member by member are based on the effects of actions calculated at $t = 0$. This is, of course, a gross simplification because any indirect effect of actions is then neglected. Such a simplified procedure is acceptable only when used in conjunction with a simplified representation of the fire environment, typically a nominal or standard time-temperature fire curve. It is accepted in that case because the representation of the fire is conventional and cannot pretend to reproduce or predict the real fire development that could take place. Then the structural analysis can also be done on the basis of a conventional situation for the effects of actions, namely the situation at time $t = 0$. The goal of such a simplified analysis is not to represent the behavior of a real structure in a real fire. Rather, the aim is to predict the result of a standard fire test that would be performed in a laboratory on a simple element subjected to a standardized time-temperature curve.

Even when the structural analysis will be performed in a continuous manner during the course of the fire, it is sound practice for the engineer to have a close look at the results of the structural analysis at time $t = 0$. This provides a good opportunity to verify whether the results obtained match the expected perception of the solution. Any structural engineer should have enough experience in structural analysis at room temperature in order to judge whether the obtained bending moment diagram and the deflection shape, for example, are consistent with the boundary conditions and loads that were supposed to be applied to the structure. This also allows for checking the stress level that exist in the structure before the fire starts, which gives an idea of the load level and, hence, of the possibility for the structure to present a significant degree of fire resistance.

In practice, the structural analysis at time $t = 0$ can be performed by an elastic analysis, because it is reasonable to assume that the structure will exhibit very little, if any, nonlinear material behavior under the design load combinations in case of fire. Indeed, if the situation that prevails at the beginning of the fire is compared to the situation that has to be taken into account for the design of the structure under normal conditions, the design values of the mechanical loads are lower, as well as the partial safety factor dividing the resistance of the material. For example, a steel structure that has been designed to sustain in normal conditions a design load equal to $1.20D + 1.60\ L$ with a design value of the material strength will exhibit very little plasticity at the beginning of the fire if the load is only $1.20D + 0.50\ L$ and the actual full material strength can be mobilized, where D is the nominal dead load and L is the nominal live load. Additional discussion of limit states design principles was covered earlier in this chapter. Because the effects of actions are determined at time $t = 0$, the stiffness of the material at room temperature is, of course, taken into account. If the structure is simple, the analysis is trivial, but if the structure is complex, it is possible to use one of the numerous numerical tools developed for the analysis of structures at ambient temperature. If the structural analysis is performed continuously during the fire course and a nonlinear method is used for that, it is simpler to use the same method also at time $t = 0$.

Another good practice before performing the analysis of the structure during the fire is to load it until failure. The loads applied in the fire situation are simultaneously and proportionally increased until collapse while the temperature is maintained at ambient. The ratio between the value of the loads in the fire situation and the value of the loads at collapse is the load level. The lower the load level, the higher the fire resistance can be. Here also, the tool of choice for determining the collapse load is the tool that will be used for the analysis under elevated temperatures.

Structural Analysis During the Fire

Except when the structure is analyzed member by member, it is common practice to take into account the effects of indirect actions and of large displacements during the fire. Thus, the structural analysis has to be performed in a continuous manner during the fire exposure.

The procedure is first to apply the loads while the structure is at ambient temperature and then to let the temperature increase in the structure while the external applied loads are usually kept constant. The response of the structure is calculated until failure and, in simple structures, this time of failure may be considered as the fire resistance time for these applied loads.

If the load-bearing capacity at a prescribed fire resistance time has to be calculated, the preceding procedure has to be repeated in an iterative manner with the applied loads being modified until the obtained fire resistance time matches the prescribed resistance time. It is possible to apply a procedure that yields directly the load-bearing capacity at the prescribed fire resistance time, as is described later.

Elastic or Elastoplastic Analysis An elastic or elastoplastic analysis for the combined effects of fire and structural loads is very rarely performed due to the nonlinearities present.

Timber constructions may be an exception, because thermal expansion in wood is normally neglected and no indirect effects of actions take place in a timber structure. A structural analysis method established for the ambient temperature situation could thus be applied in the fire, simply taking into account the fact that the stiffness of the members subjected to the action of the fire is reduced by loss of cross section, as the charring depth progresses in these members. It should be kept in mind that the effects of large displacements may increase as the fire progresses because of this reduction in stiffness of certain members, and may become crucial in the fire, whereas these could be disregarded in ambient design. The strength of the connection should be checked at every stage during the fire, as well as the decrease of the stiffness of the members and its effect on the load transfer in the structure.

One could envisage performing a thermoelastic analysis of a steel structure by one of the elastic methods normally used at ambient temperature, in which the thermal expansion and the reduction of Young’s modulus would simply be introduced in order to reflect the increase of temperature. Such a procedure would have two disadvantages: that (1) the indirect effects of actions would be severely overestimated, because in reality plasticity in the members relaxes the thermal strains, and (2) any benefit from load redistribution by the formation of plastic hinges could not be accounted for.

One could also envisage performing a structural analysis of a steel structure by one of the methods established for normal ambient conditions and based on an elastic–perfectly plastic material model. This would, at least, solve the second disadvantage of the purely elastic models, namely the incapability to consider the formation of plastic hinges. The fact that the “true” nonlinear stress-strain of the material has been approximated by an elastic–perfectly plastic behavior would still lead to an overestimation of the indirect effects of action and to an underestimation of the displacements. This is why such a procedure is rarely applied. It is considered that a structural analysis based on a full nonlinear material model is not that much more complicated and is more suitable to capture the real behavior of the structure during a fire.

One practical and interesting application of elastic or elastoplastic analysis methods for the structural analysis during the fire is in very large structures that are only partially subjected to the fire. It may, in fact, prove to be an efficient procedure to limit the full nonlinear analysis to those parts of the structure that are subjected to the fire or that are in the near vicinity of the fire affected zone, and to rely on a more simple model for those zones that are far away from the fire and deemed to behave elastically. It will, nevertheless, be necessary to verify that the obtained effects of actions in the supposedly elastic part of the structure can indeed be accommodated by the assumed elastic members. Thermal elongation of rather long portions of concrete slabs subjected to the action of two burning cars has, for example, led in 2010 to the collapse in shear of a column that was 12 m away from the fire source in the “Tour d’Ivoire” building in the city of Montreux in Switzerland, see Fig. 52.7 [22].

Fig. 52.7 Concrete columns that failed in shear due to thermal elongation of the ceiling

Nonlinear Analysis The most common procedure for the structural analysis during the course of the fire is to perform a step-by-step, nonlinear analysis of the structure. The basic equations of structural mechanics are used, but most of the simplifications that were, sometimes implicitly, used for the analyses at ambient temperature cannot be used anymore. The use of a numerical program is required.

The temperature in the structure is not uniform. The level of sophistication for the representation of the temperature distribution varies from one particular method to another and from one particular software to another. Some models have a uniform temperature in the sections, with the temperature varying from one section to the other (valid only for metallic materials). Others allow a thermal gradient, linear or nonlinear, across the depth of the section, either a steel section or a concrete slab. Some others have capability for a completely non-uniform, two-dimensional temperature distribution on the sections. Still others, more rarely, may consider a full three-dimensional temperature distribution on the whole structure.

A sophisticated constitutive model represents the behavior of the material at the local level. Most of these models are based on strain decomposition. The total strain, that is, the one resulting from a spatial derivative of the displacements, is decomposed into several components accounting, for example, for elasticity, plasticity, cracking, true creep, transient creep (in concrete), and thermal expansion. All these terms are temperature dependent, most of them in a nonlinear manner. There is, for example, no such thing as a constant coefficient of thermal expansion. The thermal expansion strain is a nonlinear function of the temperature. Most of these strain terms are also stress dependent. Different material models have been developed based on different theories such as plasticity models or damage models or a combination of both [23], some in the pure local form, and some in a nonlocal form. Different models may be used simultaneously in a single analysis for a structure made of different materials. A detailed presentation of all these models is beyond the scope of this chapter.

The finite element technique is probably the most commonly used method for solving the equilibrium equations. Displacements-based elements are commonly used, although other element types are totally suitable. Here also, some care has to be taken in the development of the particular element type for the fire analysis, or in the choice of the element type taken in the library of a program that has been written for the ambient situation if it has to be used for the fire situation. The hypothesis that the neutral axis is at midlevel of a symmetrical section is not generally valid anymore, and, for example, the usual linear expression for the longitudinal displacement field in a beam element may need to be revisited.

When the structural analysis is performed in that manner continuously during the course of the fire, the numerical program tries, at every time step defined by the user or chosen automatically by the program itself, to find a displaced position of the structure that ensures equilibrium with the applied external forces. There is no separate verification of the members. The simulation will continue as long as a position of equilibrium of the structure can be found, although some members may suffer severe distortions, high level of plasticity, or cracking. The whole structure may itself exhibit very large displacements. It is the responsibility of the user to verify whether the displacements depicted by the program are still compatible with the particular serviceability limit states for the given project. A single-level single bay steel portal frame, for example, that hangs in a catenary manner such that the beam is below ground level is certainly physically not acceptable, whereas a computer program with capability for such very large deformations and highly ductile response may see no problem in that situation. Other less trivial situations may also require the attention of the engineer and an assessment decision of the analytical results.

Whereas the example of an excessively ductile result from a computer program has been depicted in the previous paragraph, the user most often faces the opposite situation, especially in complex structures. The simulation may stop when one of the members of the structure becomes unstable, while the global stability of the structure is still maintained. Such a simulated failure produced by the inability of the software solution to converge while the global load-bearing capacity of the structure has not been exhausted is called a numerical failure. Reducing the time step used for the analysis is of no use for this problem of local failure, even though other numerical failures can be solved by the use of a shorter time step. One of the main reasons of the problems created by local instabilities has been identified in the fact that the step-by-step analysis procedure was until recently based on a series of successive, quasi-static analyses. Time was not really present in the equilibrium equations, except that it changed the material properties. A position of static equilibrium had thus to be found at any time, and such a situation of static equilibrium may not prevail during the few seconds that, for example, a member in compression buckles and its axial force suddenly drops to zero and is redistributed to adjacent members. The situation develops in a highly dynamic manner. It has been shown [18] that the numerical problems created by local failures can be significantly reduced if the dynamic behavior of the structure is taken into account in the basic equations. A significant number of the structural analyses performed in the fire situation are nowadays performed in the dynamic mode, although the final technique that would avoid all numerical failures has yet to be found.

Structural Analysis at the Required Resistance Time

It is possible to calculate directly the load-bearing capacity of a structure at the prescribed fire resistance time. This procedure may be appealing because (1) it allows expressing directly the equivalent of a safety margin in the load domain, whereas the application of the procedure described in the previous section yields a safety margin in the time domain; and (2) the numerical procedure established for the loading of structures at ambient temperature can be almost directly applied, whereas the previous procedure requires more refined theoretical developments.

The procedure is to apply from the beginning of the analysis the temperatures in the structure that prevail at the required fire resistance time. The mechanical properties of the different materials in the structure are adapted at every point of integration in order to reflect the temperature level at that point. A first loading then takes into account the thermal expansion in the materials. The external loads are then applied and increased progressively while the temperature is kept constant until equilibrium is no longer possible.

Although it may be appealing, this type of procedure is much less often applied than is nonlinear analysis during a fire. The reasons follow:

1. The step-by-step nonlinear analysis aims at reproducing the development of events in the order in which the events occur during the course of a fire, namely heating of the structure under load, whereas this procedure, which loads the structure after it has been heated, is more a numerical trick used because it may be more convenient for the designer. Yet, because the physical phenomena in play are highly nonlinear, there are no guarantees that both procedures would yield exactly the same result. In other words, if a load L yields a fire resistance time R with the first procedure, it is not certain that the load-bearing capacity will be exactly equal to L if it is calculated with the second procedure at time R.
2. If the AHJ or the engineer wants to have an idea of the safety margin in the time domain, an iterative application of calculating the load-bearing capacity at the prescribed fire resistance time is then required.
3. The complex sequence of events that ultimately leads to the global collapse of a structure can be examined and may be understood with nonlinear analysis during a fire, for example when the successive failures of

different members in the structure and the load redistribution that they generate are due to different heating rates in these members. The successive failures produced by a progressive application of the external load under isotherm conditions may be completely different and would not give any insight into the real behavior of the structure during the course of the fire.

4. This procedure is, of course, not applicable when the required fire resistance time is in the cooling phase of a real fire. If the required resistance time is sufficiently longer than the heating phase, the temperatures in the structure have significantly decreased at the required fire resistance time and the corresponding load-bearing capacity could be significantly higher than it was when the temperatures in the structure were at their maximum.

Utilization of Substructures

The structure of even a rather simple building is typically composed of tens of members linked together in a way to make a stable assembly that supports the nonstructural elements of the building, the wind, the snow loads, and the live loads. The capabilities of computers have increased tremendously in recent years, and current computer programs can analyze three-dimensional structures. It is, thus, theoretically possible to undertake the three-dimensional analysis of a complete structure; however, it has to be acknowledged that there is still a limit to the size of structures that can be practically analyzed.

For everyday practice, when time and budget constraints are important limitations, it is very rare that the entire structure is analyzed for fire effects. Instead, a substructure can be extracted from the whole system and effectively analyzed. The reason for this is to be found not only in the computer time required to run the analyses but also in the time required by the engineer to develop the computer model and interpret the results.

The quality of the results from the analysis of the substructure, defined as the similarity between the analytical results of the substructure and the total structure, highly depends on the size and on the boundary conditions imposed on the substructure, that is, at the locations where the virtual cut is made between the substructure and the rest of the structure. The decision about the model size and the boundary conditions is the responsibility of the designer. The same is also true in most structural analyses made at ordinary temperatures, but the situation is more critical for an analysis in the fire condition because of the indirect effects of actions.

The most precise results would be obtained if the rest of the structure is represented by equivalent, usually linear, springs. It may be difficult and quite time consuming to determine the stiffness of all the required springs. In most cases, the stiffness's of all the degrees of freedom that form the interface are not independent. A series of independent springs is not sufficient to represent the effect of the surrounding structure; a more or less complete stiffness matrix is required. This necessity is why it is often considered as an approximation that the variables that exist at the interface between the substructure and the rest of the structure are kept constant during the whole fire duration. These variables are—degree of freedom per degree of freedom—either a force or bending moment, or a displacement or rotation.

It is possible to define a systematic procedure that lists the different steps that have to be followed and that highlights the decisions to be taken [24].

1. The effects of actions in the whole structure must be determined at time $t = 0$ under the load combination in the case of the fire under consideration (see section "Structural Analysis Before the Fire").
2. The limits of the substructure have to be chosen. The choice is made with the contradictory objective that not only does the substructure become as simple as possible but also at the same time the hypothesis of constant variables at the boundary conditions during the fire must represent an accurate approximation of

the real situation, with respect to the thermal expansion and load paths that exist in reality. The choice of the limits of the substructure is highly dependent of the location(s) of the fire. Engineering judgment is necessary.

3. All the supports of the structure that belong to the substructure have to be taken into account as supports of the substructure.
4. All the external mechanical loads that are applied on the substructure in case of fire have to be taken into account as acting on the substructure.
5. For each degree of freedom existing at the boundary between the substructure and the rest of the structure, an appropriate choice has to be made in order to represent the situation as properly as possible. The two possibilities are:
   - The displacement (or the rotation) with respect to this degree of freedom is fixed.
   - The force (or the bending moment) deduced from the analysis of the total structure made in step 1 is applied.

   These two possibilities are exclusive because it is not possible to impose simultaneously the displacement and the corresponding force at a degree of freedom. Whatever the choice, these restrictions on the displacements and these forces applied at the boundaries will remain constant during the fire.
6. The substructure that has been defined is then considered as a new structure and a new structural analysis is performed on this new structure. Either the fire resistance is based on the effects of action at time $t = 0$ and the procedure explained in the earlier section "Structural Analysis Before the Fire" is applied, or the structural analysis is performed in a continuous manner during the fire and the procedure explained in the earlier section "Structural Analysis During the Fire" is applied, in which case the indirect actions that can develop within the substructure are taken into account. Displacements (or rotations) will appear at the degrees of freedom where the force (or moment) has been imposed whereas reactions forces (or moments) will appear at the degrees of freedom where the displacement (or rotation) has been fixed.

The necessity to perform a new structural analysis on the substructure even if the fire resistance is based on the effects of action determined at time $t = 0$ comes from the boundary conditions considered at the interface. A simple example is that of a continuous beam, uniformly loaded by p, from which one of the interior spans of length L is extracted as a very simple substructure. The bending moments determined from step 1 are approximately $pL^2/12$ at the supports, with $pL^2/24$ at mid span. If the choice made in step 5 is to consider that these moments at the supports are constant, the consequence is that the end support rotation is free and the substructure is statically determinate; it will fail as soon as the bending moment resistance on the supports decreases to $pL^2/12$ and plastic hinges are formed. If, on the contrary, the choice is to consider that no rotation can develop on the support, a new and very simple structural analysis will show that failure of the fixed-fixed beam can occur only when three plastic hinges have developed, and the bending moment resistance has decreased to $pL^2/16$ at the supports and at mid span.

Fire Resistance of Individual Members

Simple calculation models or design equations are usually able to treat the fire resistance of only individual members such as one column or one beam. Because such simple calculation models have historically been the first to be developed, it has been possible for many years to determine the fire resistance for only individual members. When complex building structures had to be evaluated with regard to their fire resistance, all constitutive members were evaluated separately, and it was traditionally considered that the fire resistance of the global structure was equal to that of the weakest member.

In fact, except for very simple structures, there is no guarantee for that conclusion. Individual members are indeed connected to each other, and there are strong interactions between their individual behaviors. As a matter of fact, these interactions are stronger in the fire than at ambient temperature because of the high levels of thermal expansion that occur at elevated temperatures. These interactions come either from additional indirect effects of actions caused by restrained thermal expansion or from large displacements caused by free thermal expansion. They can have beneficial or unfavorable effects on the fire resistance of the members and their connections and, as a consequence, on the fire resistance of the whole system (see next section, "Fire Resistance of Frames"). Therefore, this simplified member-by-member analysis and design is limited in accuracy because any changes due to the fire in load effect on the members are ignored, not only in magnitude but also in type and direction (e.g., primary bending/shear or compression load effects can change to tension [catenary action] due to thermal effects).

It is easily accepted that, because the interactions are neglected in an analysis of the fire resistance made member by member, the fire resistance of the weakest member is not exactly equal to the real fire resistance time of the whole structure. But most people think that there is at least a relationship between these two values and the correlation is positive. In other words, if the fire resistance of the weakest member is increased, the fire resistance of the whole structure is also increased. If this is true, the analysis made member by member can be used, if not to quantify exactly the time of fire resistance of a complete structure, at least to compare different systems or solutions between each other, which means that it can be used for a grading system, albeit a conventional one. A structure in which all members have a fire resistance of 2 h would necessarily be safer than a structure in which the members have only a 1 h fire resistance.

This is why more general calculation models, usually based on numerical modeling, have been developed. These allow analyzing in a more realistic manner the behavior of complete structures and determining their fire resistance as their true ability to sustain the applied loads during a certain time, and not as the minimum of otherwise independently analyzed elements.

Yet, despite its drawbacks and limitations, the analysis of structures member by member is still widely used today. It will probably continue being used for the foreseeable future and thus is worth consideration for the following three reasons.

First, the utilization of complex computer software requires a high level of education, expertise, and experience, whereas simple models for the analysis of individual members are more easily understood and applied.

Second, the costs of acquiring sophisticated numerical software, learning to use it, creating the numerical model of the structure to be analyzed, running the analysis, and interpreting the results are not always compatible with the size of the project and the resources that can be allocated to the thermal and structural analyses. An approximate conservative answer that can be obtained rapidly may be more valuable than a more precise answer that takes weeks to develop.

Finally, it has to be recognized that, for most usual structures of reasonable size and complexity, the member-by-member analysis provides a reasonable estimation of the fire resistance time of the structure. Also, in many cases, only this resistance time is to be determined, whereas the deeper understanding of the true failure mode is not required.

Most simple design equations used in the fire situation for the analysis of individual members are a direct extrapolation of the methods used for the same member at room temperature in which the stiffness and strength of the material have been adapted in order to reflect the effects of the temperature increase, although some particularities may appear in some design equations at elevated temperature. It is noted that the empirically derived correlations or tabulated data for member fire resistance based on standard fire tests are not considered to be a general structural analysis–design solution in this context and so are not addressed in this section.

Of course, the results of prescriptive fire tests for the standard fire, acceptance criteria and limited assembly conditions, together with their interpolations, can provide some validation benchmarks for the engineering methods outlined.

The design equations that give the structural resistance R_{fire} to be used in Equation 52.1 are different depending on the material type (steel, concrete, composite steel-concrete, or timber), on the type of effect of actions (tension, compression, or bending), as well as on the national building code and/or design standards to be used. The actual formulas or values for the various fire design variables, such as material strength changes with temperature, section reduction criteria, and buckling coefficients, are given by the respective design standards and building codes. Design differences among countries in this representation do exist, even though the underlying heat transfer and structural behavior are identical. The detailed equations for evaluating the fire resistance of individual linear members at elevated temperatures are given in Chaps 52–54. of this Handbook. Only floors and walls will be treated here as their design is not systematically treated in textbooks while their behavior is worth discussion.

Floors

Floor systems are usually assessed in the fire scenario where the fire is applied underneath the floor. This means that the floor evaluated is in fact the ceiling of the fire compartment. The attack from the fire that develops on that floor is usually not considered for various reasons: buoyancy that directs hot gases and diffusion flames toward the upper zone of the fire compartment, eventual presence of a mineral material that covers the floor, or presence of ashes on the floor that obstruct radiative impinging flux.

Many floor systems span in one direction only. This is the case, for example, for hollow core concrete slabs, for composite steel-concrete floors based on corrugated steel sheets, or for traditional timber floors based on simply supported timber beams. In these systems, the load-bearing capacity is assessed by the methods established for beams in bending. Additional requirements are normally imposed in order to ensure the separating function required for these horizontal elements that usually play a role in the compartmentation of the building. These requirements have to do with the thickness in concrete-based floors in order to limit the temperature increase on the upper side of the floor. In timber floors, these requirements may have to do with the thickness of the planks and also with the arrangement of the lateral joints between the planks because these joints are a weak point for the passage of hot gases.

Floor systems that span in two directions were traditionally designed in the fire situation on the basis of the bending yield-line theory, as for the ambient temperature situation. The effective yield strength of reinforcing bars in the lower zones of the slab was simply adapted as a function of their particular temperature, and the compressive zone in concrete was eventually reduced in thickness on the support lines where continuity of the slab exists. Full-scale tests performed in Great Britain [25] have demonstrated that such a design is over conservative. In reality, the load transfer system in a slab is significantly modified when the slab exhibits large deflections. In the fire, the thermal gradient on the thickness of the slab induces such high deflections already in the early course of the fire. At the later stages, when the stiffness of the slab is decreased, the applied loads also induce large deflections. In such a highly deformed position, tensile membrane forces develop in the central part of the floor, whereas a compression ring is established near the supports. If sufficient reinforcing steel is present in both directions, the loads can then be transferred to the supports more by tension in the bars than by bending.

This effect has been demonstrated in the Cardington full-scale test and has been reproduced in smaller scale but better controlled experimental laboratory tests as well as in numerical modeling. A simple method to be used by designers has been established [26], which takes that tensile membrane effect into account. When the design is performed according to this more realistic load transfer mechanism, the floor slabs can span over wider surfaces in the fire than what the serviceability limit state allows in the ambient situation. It is thus possible to leave some intermediate supporting beams unprotected.

The tensile membrane effect can develop even in systems that are considered at room temperature as spanning in one direction (e.g., in composite floors with corrugated steel deck), provided that sufficient reinforcing is provided in the transverse direction.

Walls

Walls are vertical plane elements commonly found is most building structures. Different types of walls exist, depending on the basic material and on the way they are constructed. For example, brick walls, concrete block walls, concrete walls, steel-stud or timber-stud gypsum plasterboards, and sandwich steel panels are common construction types. Columns may be inserted in the wall in order to provide lateral stability (Fig. 52.8). Walls can also be made of a combination of masonry comprised between concrete columns and beams (Fig. 52.9).

Walls can also be classified depending on the function(s) that they have to fulfill. The main functions of walls follow:

- *Separating function in normal conditions*. A wall may be used in order to separate two rooms in a building visually, thermally, and/or acoustically. Two bedrooms in the same apartment may, for example, be separated by steel-stud gypsum plasterboard that has separating functions only in normal conditions.
- *Separating function in the fire condition*. The wall is used in order to prevent the fire from spreading from one fire compartment to the other.
- *Load-bearing function*. The wall is used in order to carry some structural loads induced in it by horizontal elements that it supports such as the floors and the roof of the building.

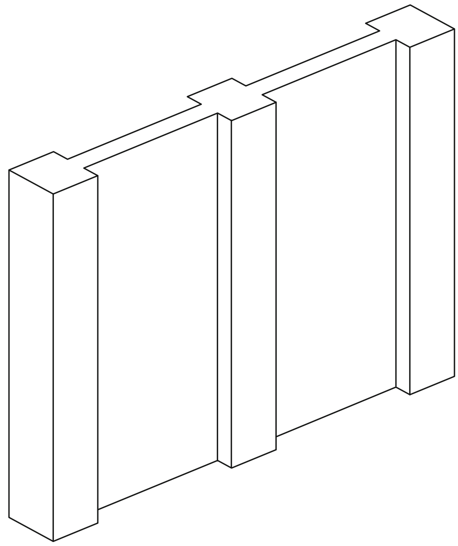


Fig. 52.8 Columns in a wall

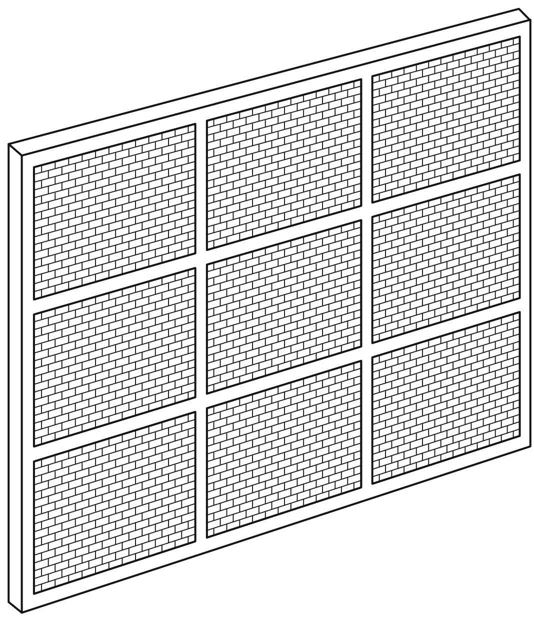


Fig. 52.9 Masonry in a reinforced concrete grid

A wall that has only a separating function in normal conditions but not in a fire is called a nonrated wall. The presence of such walls influences the development of the fire because, if only during the initial stage of the fire, the walls mark the boundaries of the room of origin of the fire and thus deeply influence the ventilation conditions in this room. Yet, the influence of such walls is normally not considered in the design fire scenario because, first, there is no means to estimate the amount of time during which their influence will persist and, second, there is absolutely neither reliability of the influence of such walls during the fire nor reliability of the presence of the wall when the fire starts.

A wall that has a separating function in the fire is usually rated by a standardized fire test. The wall is exposed to a normalized time temperature-time curve on one side, whereas the conditions on the unexposed side are monitored and compared to defined performance criteria. The temperature is monitored on the unexposed face, and acceptance criteria are imposed on the average temperature increase as well as on the highest temperature increase at any point. Radiation emitted toward the environment by the unexposed surface can also be recorded and compared to a defined level. These criteria allow verification of the insulating property of the wall. The temperatures and the emitted flux must remain sufficiently low so that they do not pose a threat to any nearby fuel (combustibles) that may be present in the vicinity of the wall or to persons who may need to use the compartment as an escape route or as a rescue area. The characteristic of integrity is also verified continuously during the fire. In the test standard ASTM E119 [2], for example, this is done by verifying that "passage of flame or gases hot enough to ignite cotton waste" will not occur near any visible crack, fissure, or opening that appears in the wall's unexposed side. Of course, even if no external load is applied to the wall, it has to carry its own weight; any collapse of the wall from the effect of its own weight automatically leads to the failure of the two previously mentioned criteria.

A wall that has a load-bearing function must support the applied load during the prescribed fire duration. In most situations, such a wall has also a separating function and is tested as exposed to the fire on one side only. It may yet occur that a load-bearing wall is subjected to the fire on both sides. This could be the case, for example, for a shear wall of limited length located within a compartment in order to transfer horizontal loads applied in the plane of the wall. Representing such a situation in an experimental test is not easy, and walls are rarely, if ever, tested with the fire on both sides.

The insulating performance of the wall can be calculated only in very simple configurations. It is, for example, generally considered that the temperature on the unexposed side of a concrete wall can be calculated with sufficient precision by numerical methods. The temperature of masonry walls can also be estimated as long as there is no macro crack in the wall. For most separating walls, however, the insulation and the integrity criteria can only be verified by experimental tests; numerical calculation provides little help. This is because the behavior of these walls made of several components strongly depends on the relative behavior of the different components. The opening of the joints between adjacent gypsum panels in a timber-stud gypsum boards wall, the behavior of connections between a steel panel and a steel stud, the charring in horizontal joints of plane timber walls, or the settlement of fiber-insulating panels inside sandwich walls each have a direct consequence on the insulation and integrity criteria. Yet, these relative behaviors can hardly be predicted for several reasons. First, these are fully coupled phenomena; the temperature field influences the deformations, but the opening of the joints influences the temperature distribution. The scale that should be considered to model these multi-physics phenomena is so small that it is absolutely incompatible with the scale of a complete building wall. Second, these behaviors exhibit a high level of variability because of the overwhelming influence of small local details such as the size and topology of a glued connection or the fact that a mechanical fastener is perfectly perpendicular to the steel sheeting or not. Also, it has to be recognized that a comprehensive constitutive model for, say, gypsum plaster at elevated temperatures has still to be established. For similar reasons, the load-bearing capacity of walls can be calculated only in the same simple configurations.

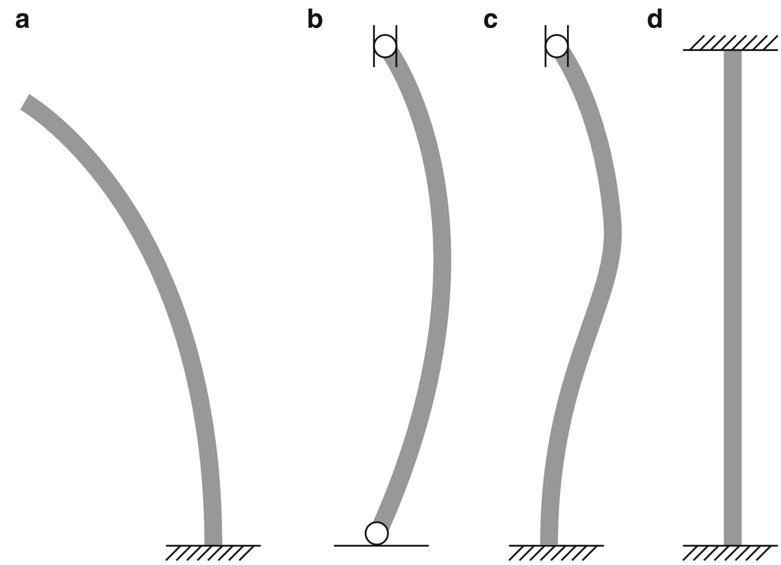


Fig. 52.10 Different lateral supports of a wall (fire on the right-hand side)

As far as the load-bearing capacity is concerned, there is no conceptual difference between a load-bearing and a non-load-bearing wall. The first one has to support an applied design load but the latter one has to support its own weight anyway. The difference is then just in the level of applied load which could affect the wall deflections, damage, integrity and stability during the fire. However, the mechanical behavior and the calculation methods for the two cases are the same.

The biggest difference between a wall and the other types of vertical elements, namely the columns, is that walls are very often heated on one side only. If a wall is heated on both sides, its behavior is similar to that of a column.

When the wall is heated on one side only, a severe thermal gradient appears across the thickness of the wall. Because of the thermal elongation in the material, this leads to a curvature in the section and, hence, to large deformations or, if the deformations are restrained, to indirect effects of actions. Four typical behaviors can be noted, depending on the type of lateral support provided to the wall. Figure 52.10 shows schematically four possibilities with regard to the lateral supports.

The wall may be simply cantilevered from the floor with no lateral support at the top (see a in Fig. 52.10). Such a wall is statically determinate and, in a first-order theory, thermal gradients will not induce a variation of the effects of actions. Yet the lateral displacements of the wall are not restrained and large displacements will indeed occur. The displacement at the top of the wall is proportional, according to a first-order theory, to the curvature induced by the thermal gradient and to the second power of the height of the wall. The displacement at the top of significantly high walls can be very important, easily on the order of several hundreds of millimeters. This important relative displacement between the fire wall and the adjacent structure that supports the rest of the building must be accommodated. The lateral displacements lead to an increase of the bending moment, especially at the base of the wall, because of the eccentricity created for the applied load and for the dead weight. This increase of bending moment will increase the displacements, which, in turn, will increase the bending moments further. The process can converge to a position of equilibrium or can lead to the collapse of the wall. Collapse can occur either because the combined effects of actions at the base of the wall exceed the resistance of the section or because the foundation has not been foreseen to withstand this increased eccentricity of the load and the rotation of the foundation cannot be prevented.

The wall may be simply supported at the base and at the top (see b of Fig. 52.10). In that case, the wall is still statically determinate and, in a first-order theory, thermal gradients will not induce a variation of the effects of actions. Lateral displacements will also occur but, compared to the cantilevered wall, they are four times smaller. Such walls are thus inherently more stable than the cantilevered walls and, furthermore, there is no danger of rotation of the foundation. The increase of the bending moment due to large displacements is much lower, and the maximum increase occurs at midlevel where only half of the dead weight is applied. A horizontal force will be applied to the supporting structure at the top of the wall because of the eccentricity of the dead weight, but this force should be easily accommodated by the structure that remains on the cold side of the wall. The challenge with this type of wall is that it has to be linked horizontally at the top to the structure on both sides of the wall. The fire may indeed occur on either side, and the support must be provided when the remaining cold structure is the one on either the left side or the right side. But, because the structure on either side is attached to the wall, this means that the collapse of the structure on the fire side may tear the wall down. Some developments have been made in order to disconnect the wall from the heated structure when it collapses; some are based on topological details that transmit the horizontal reaction only in the direction from the wall to the structure but not in the reverse direction, and some are based on plastic materials that are supposed to have melted when the heated structure collapses.

A wall that is fixed in rotation at the top (see d on Fig. 52.10) is not easily realized in practice. Such a wall would see an increase of the bending moment due to thermal gradients that are uniform along the height of the wall. Because of the fixity of the rotation at both ends, no lateral displacement would be induced in the wall, neither toward the fire nor away from the fire.

The wall shown as c in Fig. 52.10 can be constructed. It leads to lower lateral displacements than the simply supported wall B, but the horizontal force induced on the support at the top will be much higher. Such a configuration is not often used.

Structural Steel Design Criteria in the United States

In the United States, ASCE/SEI 7-10 [19] and ANSI/AISC 360-10 [21] are the fundamental design standards for structural steel and composite steel-concrete building construction. The former document specifies the design loads whereas the latter covers the structural design criteria. These documents should be utilized for any implementation of the pertinent design requirements.

It is noteworthy that ANSI/AISC 360-10 [21] includes an Appendix 4, "Structural Design for Fire Conditions," which contains provisions for both advanced and simple analytical methods, as well as acceptance of the traditional prescriptive methods based on standard fire testing.

The major characteristic of the AISC limit states design for fire conditions is substitution of the degraded mechanical properties (yield strength and elastic modulus) at elevated temperatures for their ambient counterparts, assuming elastic–perfectly plastic material response. Special provisions have been added to account for high temperature stability effects on compression members and laterally unbraced beams. Otherwise, the equations for design strength of steel and composite members remain identical to those specified for ambient conditions. The design basis fire may be a standard exposure or a postulated natural/real fire for the given space occupancy and use.

For example, under combined axial compression and bending loads, the AISC interaction equations for doubly and singly symmetric members at elevated temperatures would become the following:

$$\text{For } \frac{P_u}{\phi_c P_n(T)} \geq 0.2 : \frac{P_u}{\phi_c P_n(T)} + \frac{8}{9}\left[\frac{M_{ux}}{\phi_b M_{nx}(T)} + \frac{M_{uy}}{\phi_b M_{ny}(T)}\right] \leq 1.0$$

$$\text{For } \frac{P_u}{\phi_c P_n(T)} < 0.2 : \frac{P_u}{2\phi_c P_n(T)} + \frac{M_{ux}}{\phi_b M_{nx}(T)} + \frac{M_{uy}}{\phi_b M_{ny}(T)} \leq 1.0 \tag{52.4}$$

where P_u and M_u are the required factored axial and flexural strength per ASCE/SEI 7-10 [19] (see earlier section "Structural Load Combinations for Fire Resistance"), respectively; and $\phi_c\ P_n(T)$ and $\phi_b\ M_n(T)$ are the design axial compressive strength and flexural strength, respectively, per ANSI/AISC 360-10 [21], inclusive of the material property dependency on the high temperatures. The applied and resisting bending moments are referred to both the strong (x) and weak (y) principal axes of the cross section. In a similar manner, the remaining AISC provisions for ambient design can be readily converted to their corresponding design strength under fire exposure based on the predicted structural member temperature(s). Appendix 4 of ANSI/AISC 360-10 [21] permits the fire load analysis to be simply performed for the same member support and restraint boundary conditions as encountered at ambient.

Given the simplifications of the member by member approach, Appendix 4 of ANSI/AISC 360-10 [21] also allows, in general terms, for more advanced analyses and alternative design solutions that include effects of thermally induced deformations, framing restraint/continuity, and any load redistribution during the fire. Many of the methods cited earlier and in the following section are of this type, including those contained in internationally recognized design standards and in the technical literature. These advanced analysis methods for structural fire engineering will usually require use of computer models with the capability for material and geometric nonlinearities.

Fire Resistance of Frames

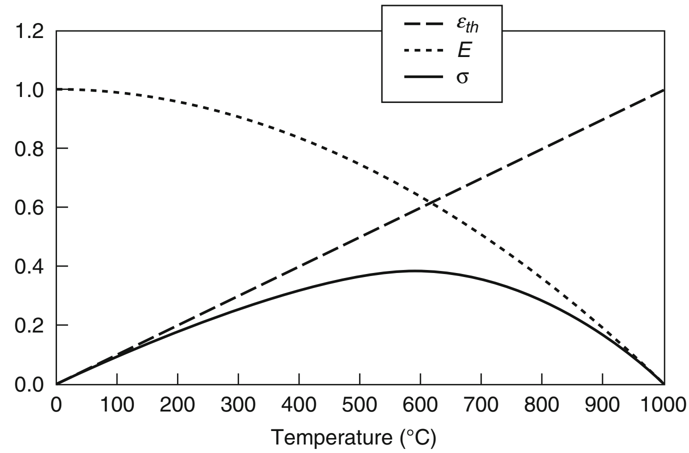
A frame is a structural system very often used in building construction. It is defined here as an assembly of several linear elements connected together to form a skeleton that extends vertically (columns) and horizontally (girders and beams) in one (two-dimensional frame) or two (three-dimensional frame) directions. The frame supports other components of the construction such as the floors, the roof, and the walls. In a framed building, the building and the frame are comprised in the same volume; the physical space that exists between the different elements of the frame is the usable space of the building.

Although the discussion in this section will be limited to two-dimensional frames for reasons of simplicity, most of the concepts highlighted here can be extended to a three-dimensional configuration.

Indirect effects of actions appear in a structure during the course of the fire because of thermal expansion. This is at least the case when the elements are made of metal or concrete but not so much for timber elements because this material exhibits very little thermal expansion. The indirect effects of actions come from the combined influence of two opposite sources:

1. Geometrical second-order effects, created by the change in position that the thermal expansion produces in the structure. It has been explained, for example, how severe this effect can be in a cantilevered wall (see the section "Walls" earlier in this chapter). If a high-rise building would be affected by a fire on several floors but on one side only, the situation could develop in a similar manner. Structural elements subjected to an axial force will show an increase of bending moment if lateral displacements are created in the elements by an unsymmetrical temperature distribution in the section. Similar effects are also produced if the displacements are caused by a decrease in stiffness of the elements.
2. Thermal expansion that cannot develop freely in an element will also induce variations in the

Fig. 52.11 Evolution of the restraint stress with the temperature



effects of action. When a column cannot elongate, an increase in axial force is induced in this element. Because of the variation of axial force in, say, a beam, the columns connected to that beam will consequently see an increase in bending moment. Similarly, a beam subjected to thermal gradients across the thickness will see a variation of the bending moment distribution if the transverse displacement that would be generated is prevented; this is the case, for example, of a continuous beam on more than two supports or of a beam in which the rotation is fixed at the supports.

These combined effects usually are detrimental to the fire resistance of the frame but, in some circumstances, can have a neutral or beneficial influence. This is the case, for example, when tensile forces can be mobilized to withstand the loads that bending cannot accommodate anymore.

If the design fire scenario is based on a nominal fire curve in which the temperature increases continuously, or if failure of the structure occurs during the increasing phase of a more realistic fire scenario, indirect effects of actions created by large displacements have a tendency to increase constantly during the fire. This is because the temperatures of the parts of the structure subjected to the fire increase constantly and, as a consequence, the stiffness of these parts decreases constantly, which leads to a continuous increase of the displacements.

The situation is more complex when the indirect effects of actions are due to restrained thermal expansion. This can be explained quantitatively by the model of a simple element that has a thermoelastic constitutive model and has the elongation totally restrained. The increase of stress $\Delta\sigma$ due to a thermal expansion $\Delta\varepsilon_{th}$ is equal to the amount of thermal expansion that cannot develop. For this case, it equals the whole thermal expansion (because the restraint is total) multiplied by the Young's modulus of the material E in Equation 52.5:

$$\Delta\sigma = E\varepsilon_{thermal} \tag{52.5}$$

Owing to the fact that thermal expansion is usually an increasing function of the temperature and that the Young's modulus is a decreasing function of the temperature, the product of these two variables first increases, passes through a maximum, and then decreases again. This is illustrated schematically by Fig. 52.11 in which the supposedly linear thermal strain has been normalized to 1.0 at 1000 °C and the nonlinear decrease of the modulus E has been normalized to 1.0 at 0 °C. In that hypothetical case, the so-called "thermal stress" would have a peak around 580 °C. If the restraint is not total but the restraining structure behaves elastically, the

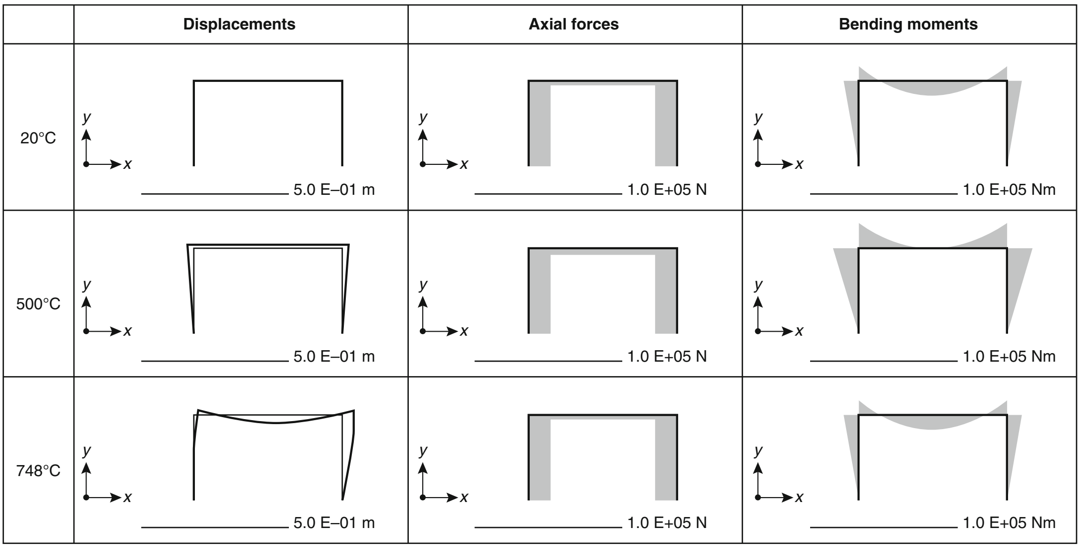


Fig. 52.12 A simple portal frame under increasing temperature

thermal stress will be lower in magnitude, but the maximum will occur for the same temperature of the heated parts.

Even if all elements of the frame are subjected to the action of the fire and in the hypothetical case that all elements have their temperature increasing at the same rate, a restraint to thermal expansion is likely to appear. This is because the supports of the structure on the ground (that is, usually, the bottom of the columns of the first floor) are normally restricted in their horizontal displacement.

Figure 52.12 shows the evolution of the displacements and the effects of actions in a small simple portal frame, 5 m wide by 3 m high, the steel members of which are heated uniformly. If the effects of creep are neglected, the situation can thus be depicted as a function of the steel temperature, independently of the fire scenario. The beam is subjected to a downward uniformly distributed load of 30 kN/m, with an associated horizontal distributed load of 30 N/m introduced as an equivalent initial imperfection. The section on the beam is an IPE400 and the section of the column is HE280B. Steel has yield strength of 235 N/mm^2. The results have been obtained by a nonlinear numerical analysis that takes large displacements, thermal expansion, and nonlinear stress-strain relationships into account.

At room temperature, the displacements can hardly be seen on the picture, although they have been amplified by a factor of 5. The axial force and bending moment diagrams are standard for this type of structure.

When the temperature of the elements increases, the thermal expansion of the columns develops freely, while the thermal expansion of the beam is restrained by the bending stiffness of the columns. The compressive axial force in the beam increases, accompanied by the same increase in horizontal reaction force directed toward the inside of the frame at the two supports. This induces a significant modification in the bending moment diagram.

The axial force in the beam increases until 500 °C and decreases thereafter. It has to be noted that, in this case, not only the stiffness of the restrained member (the beam) decreases as the temperature increases, but also the stiffness of the restraining system (the columns) does.

For temperatures beyond 500 °C and until failure at 748 °C, the indirect effects of actions decrease continuously, and at failure the effects

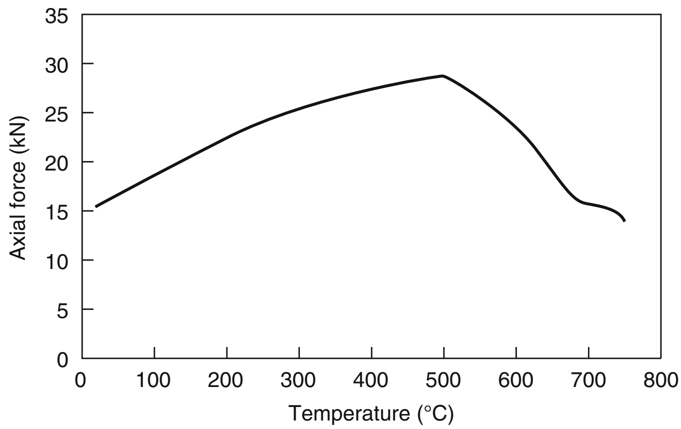


Fig. 52.13 Evolution of the axial force in the beam

of actions have nearly the same pattern as at room temperature. This behavior has been observed on many occasions, especially when the failure temperatures are rather high.

Figure 52.13 shows the evolution of the axial force in the beam as a function of the temperature in the structure. It shows that, although the force has approximately doubled at 500 °C, its value at failure is nearly identical to the value it had at room temperature.

The following example is based on a multistory, moment-resistant frame in which only one floor is subjected to the fire. The vertical distance between the beams is 3 m; and the columns are separated by 6, 8, and 6 m. The sections and the material model are the same as for the previous example. The steel columns at floor 4 as well as the steel beam that they support are heated at a uniform temperature (Fig. 52.14).

The axial force and bending diagrams at room temperature are typical for this type of structure. The displacements can hardly be seen in Fig. 52.14, although they have been amplified by a factor of 5.

During the first phase of the fire, until a steel temperature of 405 °C, the longitudinal elongation of the heated beam is restrained not only by the bending stiffness of the heated columns that support this beam but also by the bending stiffness of the cold columns just above the beam. A compression force thus develops in that beam with, for equilibrium reasons, tensile forces in the beams directly above and below the heated beam. This is reflected in the bending moment diagram of the columns, especially the outermost ones.

From then on, the compression force in the heated beam decreases, not only because of the decrease of axial stiffness in that restrained beam and in the restraining heated columns but also because of the vertical downward deflection of the beam that provides some relief geometrically to the compression force. Whereas indirect effects of actions were mainly induced by restrained thermal expansion in the first phase of the fire, large displacements will play an increasingly prominent role.

At 633 °C, failure is imminent. A severe downward deflection in the central span of the heated beam is observed. The axial force and bending moment diagrams are not as disturbed as they were at 405 °C but are still not totally equal to the pattern displayed at room temperature.

Total collapse of the frame finally occurs at 634 °C by buckling of the two central columns. The axial force diagram shows that the compression force has nearly totally vanished in the upper part of these two central columns. This is because dynamic effects have been taken into account in this analysis (heating rate = 1 °C/s). An analysis made by a succession of static equilibrium would probably stop one or two degrees earlier, thus with only a marginal difference in critical temperature but a much less complete insight into the failure mode.

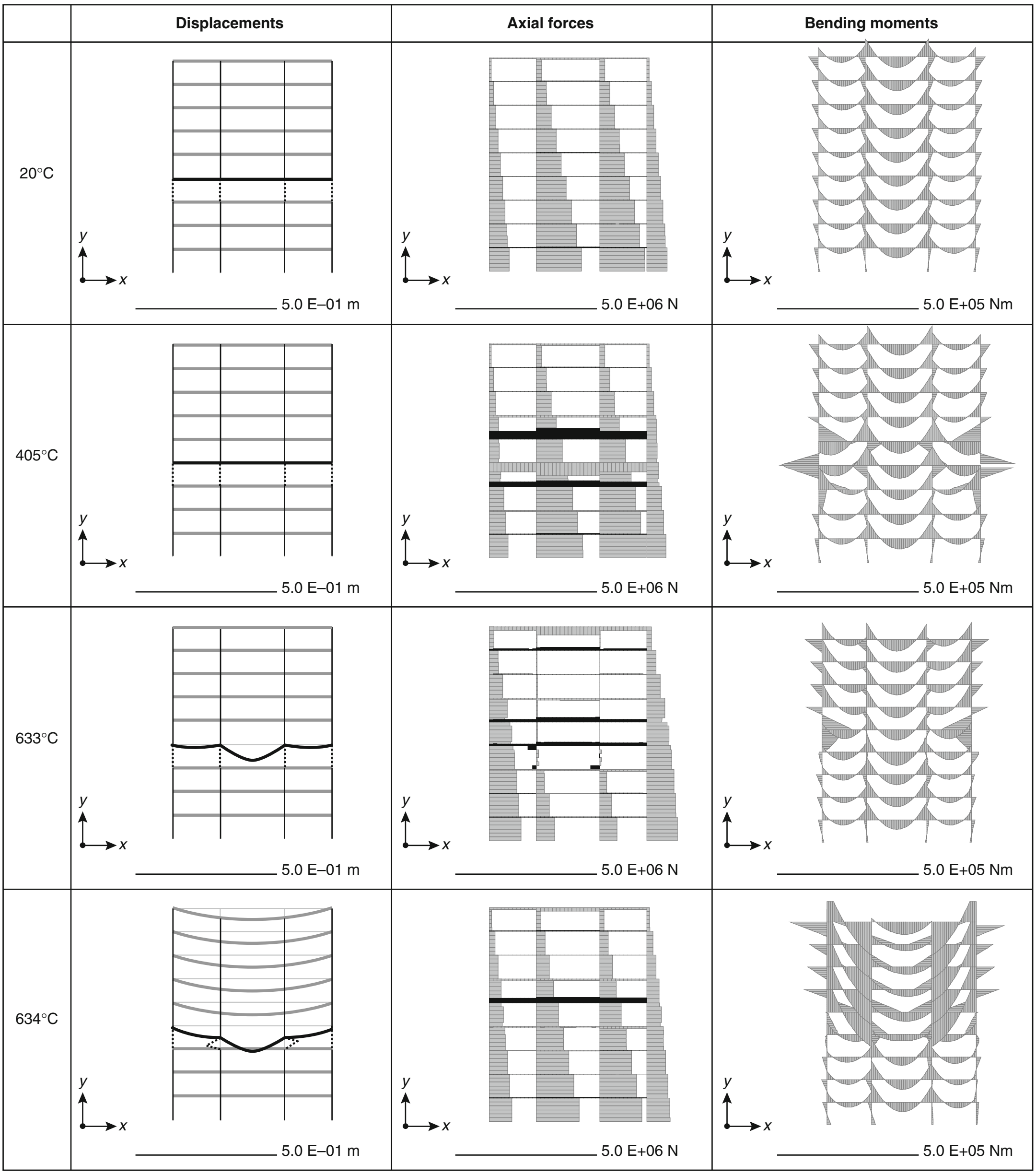


Fig. 52.14 A multistory portal frame under increasing temperature

Figure 52.15 shows the evolution of the axial force at mid length of the heated beam. It shows that, when failure is approaching, the axial force tends to decrease to the same level that it had been at room temperature, with even a sign reversal at the very end, when large displacement effects become predominant.

When the same examples are run with the thermal expansion of steel being artificially turned off, the simple frame of Fig. 52.12 fails exactly at the same critical temperature of 748 °C, whereas the multistory frame of Fig. 52.14 fails at a critical temperature of 613 °C, compared to a value of 634 °C with

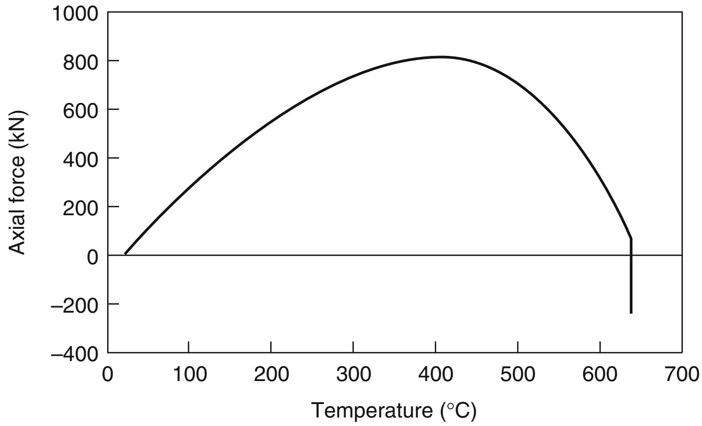


Fig. 52.15 Evolution of the axial force in the heated beam

thermal expansion being considered. This means that, in this particular example, the effects of thermal expansion are somewhat beneficial for the fire resistance.

Input and Modeling Uncertainties

Engineers and scientists are continually working with the reality of the physical world in which we are living. In particular, civil/structural engineers try to understand and quantify the different natural and materials phenomena in order to design safe structures and to protect these structures against any foreseeable loads and actions, including fire. The only way to represent the physical reality is to create analytical and experimental models that best represent it. If the models are realistic and simple enough, the engineer can vary the input of the models and see how the output varies. This allows the engineer to see the influence of different parameters on the behavior and safety of the structure.

Because the models are simplified representations of reality, they have their inherent limitations and uncertainties. Limitations are defined here as approximations that are clearly identified but accepted because of the range of input or response assumptions, or because they are considered to have a negligible influence on the results. On the other hand, uncertainties are defined as a lack of knowledge about a particular physical behavior (model uncertainty) or about the precise value of an input parameter (input uncertainties).

There are essentially two different types of model for representing reality: experimental models and analytical models. Experimental models consist of physical specimens built to represent the real structure. Very often, only a part of the structure can be represented by a specimen and tested as either a full-scale or a small-scale model. In the latter case, it is possible to represent a larger part of the structure, possibly the whole structure. Full-scale experimental models of complete structures are extremely rare in general and for structural fire response, with the one major exception and example being the series of tests performed by BRE at Cardington [25].

Analytical models are defined here as models made of equations and criteria, from the most fundamental force equilibrium, structural mechanics, and design criteria equations used in engineering office practice to the most complex numerical models (finite element), which require huge computing capabilities. A particular family of the analytical models consists of the empirical models, sometimes called the tabulated data. They are not real behavioral models, but the presentation in a simple form (data tables or statistical regression equations) of the results obtained by application of the experimental or analytical models.

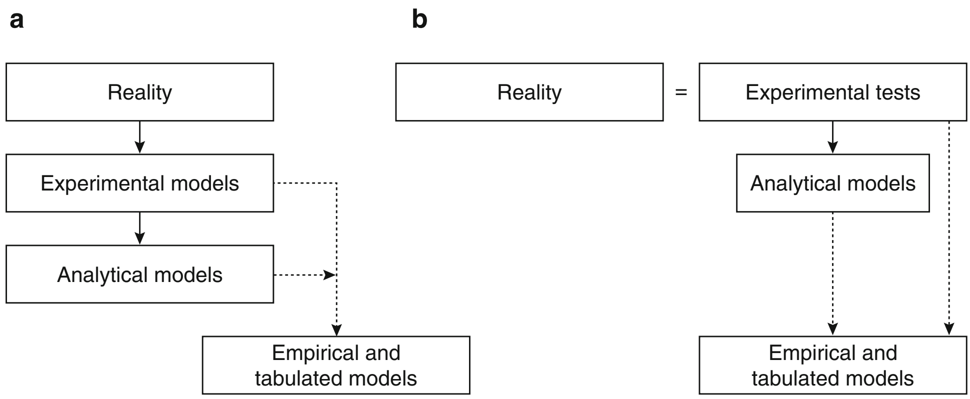


Fig. 52.16 Two common perceptions of the hierarchy in structural fire engineering

It is essential that the limitations and uncertainties of the models are fully understood by the users. If not, there is a high risk that the models may be used in an inappropriate manner or used beyond their intended scope of application. Sensitivity analyses by variation of suspect inputs can provide a more robust, bounded answer, but this is possible only if the uncertainties have been identified.

For some sciences, such as astronomy, experimental models are not feasible and only analytical models can be developed. In the field of structural fire engineering, the first experimental fire tests made on simple elements were conducted in the early 1900s before even the simplest analytical models could be created. At the end of the twentieth century, analytical models were becoming more complex due to the growth of the large historical fire test database and computational capabilities. The initial objective at the time when many of these numerical models appeared was to efficiently complement or replace fire testing of simple elements. Because of this delayed appearance of analytical models, there is a common perception that a hierarchy exists with reality at the top, the experimental models just underneath, and the analytical models at the bottom (Fig. 52.16a). For some, the experimental tests are even incorrectly considered as an exact representation of reality, not as a model of the reality (Fig. 52.16b).

The situation, that is, the relationship between the models and reality, is in fact different. First of all, it has to be realized that, when an engineer has to protect a future building against the effect of possible fires, there is no such a thing as "the reality" (Fig. 52.17).

The fires that occurred in the past provide us with some useful information but it has limited applicability. First, the historical fires may not have occurred on exactly the same structure as the one that is envisaged. Second, and above all, a building is very rarely instrumented with recording devices when a fire starts, and thus it is not easy to derive quantitative conclusions from the general observation of buildings devastated by a fire. The future fire that may occur in the building that is being designed does not belong to reality either, at least not at the time when the design is being made.

Note that one exception to this is when the case that is investigated is a real fire that occurred in a real building, in forensic investigations, for example. In that case, the text in the upper rectangle of Fig. 52.17 should be replaced by "Fire under investigation," and it belongs, indeed and unfortunately, to reality.

Second, regarding the relationship between the models and reality, if it is well accepted that analytical models, and especially the numerical models, have their own limitations, the general perception is that experimental models are a perfect representation of the real building or at least of the element under investigation. The technicians and engineers in charge of experimental fire tests know all too well this is not

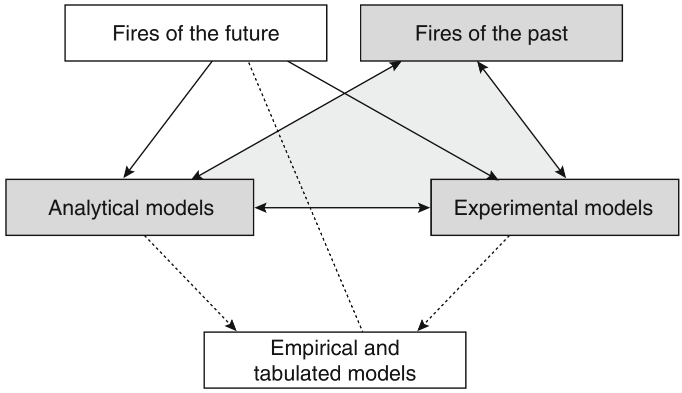


Fig. 52.17 A more correct representation of the relationships between the models

exactly the case. As it is sometimes said in a caricatured manner: "Nobody believes in the results of a numerical model, except the one who made it, but everybody believes in the results of an experimental test, except the one who made it."

Fires of the past, analytical models, and experimental models must be considered as three tools of similar utility, each with its own merits and limitations. They all should be considered on the same level, with no particular one being predominant. All three benefit from the others, as they mutually interact with each other. When analyzing a real fire, analytical models may help trying to explain what happened, and experimental reconstruction may help verifying some hypotheses made about the behavior of the building. When developing analytical models, experimental tests on simple elements are considered as the necessary point of comparison for verification, and comparison with the outcome of real fires may be considered as a good validation. Numerical models are very often used to make the predesign of experimental tests in order to ensure a higher probability of success or in a complementary manner in order to maximize the information obtained from the test.

The main limitations and uncertainties of the models will be briefly mentioned in the following sections. Most of the structures subjected to fire are designed according to the hypothesis of an uncoupling between the mechanical and the thermal problems. The temperatures in the structure are first calculated in the structure without taking the stress level into consideration, and these temperatures are then taken into account in the subsequent mechanical analysis. The limitations related to the determination of the temperature in the structure by analytical models will be discussed first, then the limitations related to the mechanical analysis by analytical models, and finally the limitations of experimental models.

Limitations and Uncertainties of Thermal Calculations

Thermal Properties In order to determine the evolution of the temperature in a structure, the thermal properties of the materials present in the structure must be known.

Generic properties are given in Appendix 2 of this Handbook for the most commonly used building materials such as structural steel, normal strength concrete, gypsum, and so forth. They can be used with a reasonable level of confidence because they have been widely validated and have been in use for several years already with no apparent significant problem. Attention must be paid when the field of application is extended to "similar" materials that may or may not have markedly different properties such as iron steel, stainless steel, high-performance concrete, bricks, mortars, and the like. The ideal situation is when the person determining the temperatures in a structure for a practical application has access to experimental results made on elements comprising the material that will have to be taken into account in the application. It is then possible to determine by trial and error the thermal properties of the material that allow reproducing the results of the tests, and these properties can be used in subsequent analyses on structures that will not be tested but only calculated. Not only the thermal conductivity and the specific heat have to be determined but also the properties for the boundary conditions, such as the convection heat transfer coefficient and the emissivity. If this cannot be done and thermal properties of similar materials are utilized, the engineer must be aware of the inherent degree of uncertainty introduced in the final results.

For other materials, mainly the thermally insulating products, no generic properties can be given and the information must necessarily be taken from experimental tests. It must be emphasized that most of the thermal properties are highly temperature dependent and a value given for a commercial product for utilization at room temperature (e.g., insulating the walls of houses against heat loss to the outside atmosphere) cannot be used directly in a fire. The thermal conductivity, to name only one, has a tendency to increase significantly with increasing temperatures.

If a producer is able to deliver for his or her product the laws giving the evolution of the thermal properties as a function of the temperature, there is a high probability that these laws have been derived by recalculations of experimental tests by a simplified calculation method in which several hypotheses are included: a uniform temperature in the section, the temperature on the outside of the insulating layer equal to the temperature of the fire, and so on. The given laws are thus suitable for utilization in the same type of simple calculation model. If numerical calculations have to be performed, it is better first to recalculate all the available experimental tests by the use of the numerical model. Because the same simplifications are not present in the numerical analysis, the laws obtained by that method could indeed be slightly different from the laws obtained with the simplified method. It is important that the same model be used for a practical application as the one used to determine the thermal properties.

Fixed Geometry In most if not all methods used for determining the temperatures in structures exposed to fire, the geometry of the sections is given before the calculation starts, and it remains fixed during the whole simulation. Theories and numerical algorithms do exist for calculating temperature distributions in objects with a shape that is continuously changing (e.g., in the ablation process that occurs at the nose of re-entrant space vehicles), but these techniques are not commonly used for designing buildings subjected to fire.

A first situation of changing geometry is the case of intumescent painting, usually applied on steel members. Whereas the dry film has a very limited thickness in the order of some millimeters, the product exhibits an endothermic chemical reaction when heated and expands to a layer of foam-like product with a thickness of several tens of millimeters. Very few attempts have been made to model precisely this expansion (see Butler et al. [27]). The usual procedure is to model the intumescent painting as a purely conductive layer of constant thickness (e.g., the thickness of the dry film) and to determine "equivalent" thermal properties yielding the same temperature evolution for the steel section as the one observed in experimental tests. A peak can be introduced in the curve of specific heat in order to account for the endothermic chemical reaction. It has to be noted that different laws of thermal conductivity should possibly be used for different thicknesses of the dry film because it has been observed that the thermal resistance provided by these products is usually not proportional to the thickness of the dry film.

Another situation is found in timber sections that exhibit shrinkage and cracking after charring. A practical solution very often used is to also consider a constant geometry and equivalent thermal properties.

The phenomenon of concrete cracking and spalling, another situation of evolving geometry, is usually not taken into account when modeling concrete structures subjected to fire. This is acceptable only for materials that are known for not being particularly prone to spalling (e.g., normal-strength concrete with a limited level of free moisture), or if particular provisions have been made in order to prevent the occurrence of spalling (e.g., the addition of polypropylene fibers). In other cases, and especially for high-strength concrete, the behavior of the material against spalling must have been determined experimentally. In case of doubt or for critical situations, it is still possible to neglect from the very beginning of the calculation the presence of the outer layer, this one being assumed to disappear in the early stages of the fire because of spalling. The question is then to decide on the thickness of this sacrificial layer. A usual choice is to limit the spalling to the first layer of reinforcing bars, these being then directly exposed to the influence of the fire, but there is experimental evidence of cases when the spalling did progress beyond the first external layer of reinforcing bars.

Equally unpredictable by analytical models at this time are delaminations or detachment of spray-on materials or gypsum board and other fire-related damage that will accelerate thermal penetration into the structure.

Perfect Contact If two materials are in contact with each other in a section, the usual hypothesis for an analysis is that the contact at the interface between both materials is perfect. This is not always the case, and sometimes initial contact does not persist and disappears during fire tests.

For example, when a concrete slab is cast in situ on the upper flange of a steel beam in order to form a composite steel-concrete section, it is generally accepted that the contact between the concrete slab and the steel beam is nearly perfect and will remain so during the fire because of the eventual shear connectors fastening the slab to the beam. If not, simply the action of the gravity load is assumed to force the slab to follow any downward movement of the beam on which it is laid.

A situation with a similar geometry can arise if a concrete facade element is placed against the external flange of a steel column. In that case, it is not possible to be sure that no gap will be created during the course of the fire between the steel column and the wall element, with each one having its own thermal and structural bowing. If at least a reasonable amount of connectors is provided, it is reasonable to take the effect of the wall elements into account by supposing that the external flange of the steel column is not attacked by the fire. On the other hand, because of the uncertainty of the contact with the concrete element, the concrete element will not be represented in the thermal analysis of the steel column, which will inhibit in the model any heat sink effect from the column to the wall (an adiabatic boundary condition is imposed on the external flange of the column).

In composite steel-concrete slabs using unprotected thin steel decking, it has been observed in experimental tests that the steel sheet very often detaches from the concrete slab after a short duration. The research work performed by Both [28] allowed to take this effect into account in a method presented in Eurocode 4 [29], but this method is an empirical method. This effect is usually neglected in numerical analyses, with the consequences that the temperatures in the steel decking are slightly underestimated. The bare steel deck loses its strength very quickly anyway, and the temperatures in the concrete slab are somewhat overestimated, which is on the safe side.

A similar situation exists in hollow steel sections filled with concrete. The external steel tube has the highest temperatures in the section and exhibits the highest radial thermal expansion. This, plus the effect of the steam pressure from the evaporating water of the concrete, leads to a far from perfect contact, and a thermal resistance does appear at the interface between the concrete core and the steel section. As it was observed that the temperatures measured in the center of the section did not compare well with the temperatures computed on the base of a perfect contact, it has sometimes been tried in the past to correct this effect by introducing very high levels of evaporable water in the concrete. Levels as high as 10 % in weight are reported in Eurocode 4 [29] although a more reasonable value of 4 % is recommended in the absence of data. It is by far physically more correct to include a thin layer of conductive material between the concrete core and the steel section. In Renaud [30], this contact resistance has been estimated to 0.01 m^2K/W, that is, equivalent to a 16 mm layer of concrete with a thermal conductivity of 1.6 W/mK (but it is geometrically more correct to use a 1 mm layer of a material with a conductivity of 0.1 W/mK)

Effects of Localized Fires The assumption most often used for the design fire scenario utilized for verification of a structure under the effects of fire is a uniform temperature in the compartment. The reasons are probably that, first of all, this is the situation prevailing in the standard fire tests that the numerical programs tried to mimic in the early days and, second, because it is a good approximation for fully developed fires in small compartments, which are the most challenging for the structure. Yet, more and more attention is being paid nowadays to localized fires, which include either any fire in its early stage or a fire in which the available fuel load is concentrated on a limited part of the floor area, such as a registration desk in an otherwise empty atrium or one car burning in a parking garage with no propagation to adjacent cars. These situations have to be considered because of the susceptibility of statically determinate structures to the failure of a single element that could be located just above the localized fire source, and also because this period of time when the situation is still tenable in the compartment is very critical for the evacuation of the occupants and for action of the fire brigade. With these localized fires, the temperature is far from uniform in the compartment and much more of a problem for the numerical programs. The temperature of the gas in the vicinity of the structure is by far not the most important parameter driving the heat transfer to the structure, with radiation from the fire source usually being dominant.

First, consequences are conceptual. The boundary conditions for the determination of the temperature in the structure are much more complex. They are certainly varying with the location of the boundary, and this possibility has to be taken into account in the numerical program. A steel beam located above the fire plume is not subjected to the same thermal attack as a beam located several meters away. Second, the question arises whether an uncoupled determination of the temperatures is still valid. Is it admissible to calculate the temperatures in the compartment using, for example, computational fluid dynamic (CFD) software and, afterward, the temperatures in the structure based on those calculated in the compartment? Or is it necessary to make a fully coupled determination of the temperatures, simultaneously in the compartment and in the structure, taking into account precisely the interaction between the two? The question has even been raised whether the mechanical analysis should not also be part of the same simulation. For example, the deflecting ceiling would modify the interior geometry of the compartment and this would also affect the development of the fire. Whereas the last coupling can probably be neglected in most cases, the thermal coupling from the localized fires to the structure must be considered, and this is not done easily.

A practical problem in which the interaction between the compartment and the structure has to be considered is the difference in size of the meshes used in both problems. Whereas the dimension of the cells can be in the order of 10–50 cm when modeling a compartment, the thickness of the web of a steel beam can be as low as 4 mm. It is not realistically possible to decrease the size of all cells of the compartment down to the size required for a precise determination of the temperatures in the structure. Algorithmic solutions must be derived in order to cope with this geometric discrepancy at the interface.

Fig. 52.18 A numerically very large structure

If the structure is subjected to the effects of a localized fire, the temperature distribution inside the structure is inherently a three-dimensional distribution. The precise determination of such a temperature distribution would normally require the utilization of a full three-dimensional model of the structure made of three-dimensional solid elements. The size of such a model would be so huge that it would not be practically feasible. Some approximations have to be made in the model, introducing some additional limitations.

Limitations and Uncertainties of Mechanical Calculations

Very Large Structures Although the geometrical dimensions of a structure are not a problem for numerical modeling, there is still a limit to everything, including the power of our processing units and the patience, schedules, or budgets of the users. Some complex steel structures may easily comprise tens of thousands of elements of different sorts, each experiencing a different temperature history (Fig. 52.18). These structures are not only geometrically big; they are above all numerically big. It is, for example, beyond any reasonable expectation to think that the whole structure of one of the WTC towers could in the foreseeable future be modeled completely in detail, with every member, every slab, and every connection represented. One possible solution in case of very large structures is to limit the analysis to substructures, that is, representative parts of the whole structure. It is also possible to represent with simple elements, possibly elastic ones, the parts of the structure that are far away from the zone affected by the fire and that are expected not to exhibit any nonlinear deterioration.

The most detailed beam finite elements used in numerical modeling allow determining the precise extent of plasticity at every point in the sections and along the members and provide a very precise shape of the deformed members; but several elements are required in order to represent each subassembly, such as a beam or a column. More simple formulations can be used, in which each subassembly is represented by a single element, for example, using the beam-column plastic hinge approach such as in Liew and Ma [31] or Landesmann [32].

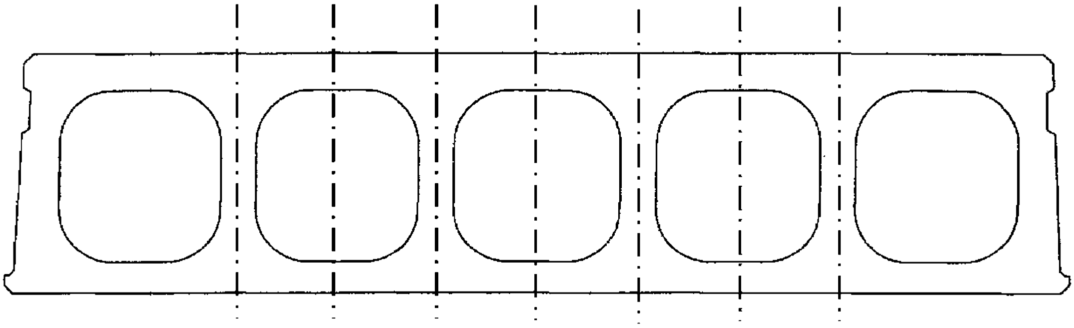


Fig. 52.19 Planes of symmetry in a hollow core slab

It has also to be realized that modeling a building structure, subjected to fire or not, using solid brick elements is far beyond our present possibilities. Only parts of the structure can be analyzed with these types of finite elements. In this case, the limitation explained in the following section will arise, namely the decision on the boundary conditions.

Boundary Conditions The boundary conditions may pose some problems in the discretizations that are particular to the fire. Some are inherent to the three-dimensional stress level in solid-type structures and they cannot be solved easily; others appear in bar-type structures when defining a substructure and selecting the appropriate boundary conditions.

Let us imagine that a simply supported flooring system made of parallel hollow core prefabricated concrete elements has to be modeled and that solid brick elements are used. If the loading is symmetric with respect to the longitudinal axis of the elements, only half of the span of the system needs be modeled. The boundary conditions along this plane of symmetry are standard and not particular to the fire. In the other direction, it is clear that a series of parallel vertical planes of geometrical symmetry do exist; for example, passing through the center of the cavities and also passing through the center of the webs between the cavities (Fig. 52.19).

If the loading is uniform on the floor, one might be tempted to discretize only a part of the structure, limited by a cavity and a web plane of symmetry. The question then arises about the boundary conditions, especially for the displacements perpendicular to the parallel planes of symmetry. If they are left free, no thermal stress will arise in this direction, and this is not correct in the fire situation. If they are completely fixed, a full restraint will be created, and this is also not correct. Even if the slab is laterally not restrained at all, a linear constraint relation between all the nodes located in a plane of symmetry would imply that a Bernoulli condition has been imposed, and this also may not correspond to reality. In fact, in order to obtain a realistic answer, it is impossible to consider all these parallel planes of symmetry. Only the one in the center of the slab can be taken into account. This means that as much as one-quarter of the whole floor has to be discretized. This can prove to create a model that is numerically very big.

In big structures made of bars, the concept of substructure is often used. In this case also, a choice has to be made for the boundary conditions at the interface between the structure and the rest of the structure (Fig. 52.20). The choice is for each degree of freedom between imposing a fixed displacement or imposing a force and leaving the displacement free. In fact, the real boundary conditions are intermediate between these two extreme solutions, dictated by the response of the surrounding structure.

Spalling The same limitations exist during the mechanical analysis as during the thermal

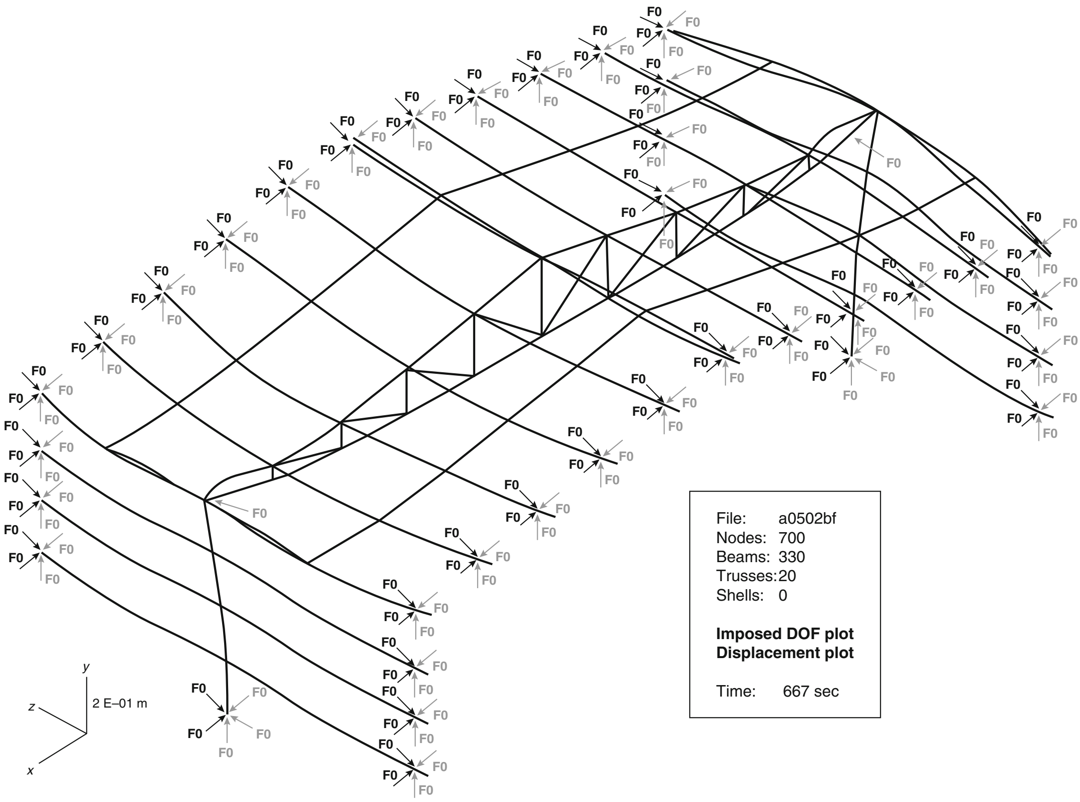


Fig. 52.20 Boundary conditions in a substructure

analysis; if the concrete spalling phenomenon is not taken into account when determining the temperatures, it is also not taken into account in the mechanical model. All comments made above in the section on fixed geometry could be repeated here.

It has to be mentioned that some efforts have indeed been made in order to predict the phenomenon of spalling. This modeling relies on highly sophisticated constitutive and numerical models taking into account the coupling between the mechanical and the thermal problems; mechanical stresses created by applied loads and by thermal restraint interaction with the water pressure. These models provide a unique insight in the phenomena that can help understand the physics and identify the relevant parameters. However, these models have to be provided with a very large number of temperature-dependent input data and the prediction cannot yet provide a complete level of confidence with respect to a phenomenon that is not really deterministic. As a consequence, in a real design situation, it is probably faster, much cheaper, and perhaps more reliable to make a simple experimental test aiming at identifying the susceptibility to spalling for a particular concrete mix/structure situation than to make all the experimental tests that are necessary to feed the model and then try to predict whether spalling is likely to occur or not.

Lack of Convergence The equations that govern the equilibrium of a structure subjected to fire are highly nonlinear, the reasons being in the geometrical as well as in the material behavior. Moreover, these equations express the equilibrium at a given time and, in order to model the evolution of the structure during the course of the fire, they have to be integrated over time. The integration of these nonlinear equations involves an iterative procedure and experience has shown that, in structures with some level of complexity, convergence of this process is not guaranteed. Depending on the size of the time step or on the value used for the convergence criterium that is used, the simulation of the same case may stop running at different moments in the fire. The simulation may stop at an early stage, when the load-bearing capacity of the structure is not yet exhausted. These failure times produced by a lack of convergence are called *numerical failures*. In some cases, the experience of the user with nonlinear modeling in general and in the computer code used in particular allows the user to find a solution that makes the code run until the real physical time of collapse. But in the most difficult cases, all the algorithmic resources are not sufficient to solve this difficulty; the simulations stop prematurely, and this is certainly a severe limitation of the numerical models.

The biggest danger is that the user may be tempted to modify, in fact to alter, the model in order to facilitate the convergence. Modification of the constitutive models is a common way to achieve this result, with creation of numerical materials that are excessively elastic or with unlimited ductility or, for concrete, with an artificial tensile strength never encountered in reality. The model will then perhaps converge, but to a solution that has an unknown relation to the solution of the original problem. The user should perhaps have the courage to admit that the tool does not do the job that was expected, rather than draw conclusions from an altered model.

There is another problem linked to numerical modeling, which is in fact nearly at the opposite of the problem of numerical failures. In some cases, the deflections of the structure can be so high when the simulation stops that they have no possible physical existence. For example, the vertical deflection of a simply supported beam could exceed the floor to ceiling distance available under the beam. Or the horizontal displacement of a rolling support could reach several hundreds of millimeters.

Because of these two opposite problems, the moment of the last converged time step cannot be considered automatically as the fire resistance time of the structure. The displacements and the evolution of the displacements during the fire are the best indicators to decide about the fire resistance time.

If the displacements at the last converged time step are exceedingly large, it is necessary to observe the displacements at previous time steps and to decide when the fire resistance of the structure was exhausted. Conventional and sometimes arbitrary failure criteria have to be used, for example, based on deflection or deflection rate limits.

If no large displacement is found, it is necessary to judge whether this corresponds to a numerical failure or to the real loss of load-bearing capacity of the structure. Finding one degree of freedom in the structure for which the evolution of the displacement as a function on time shows a vertical asymptote near the end of the simulation is a good indication of real failure (runaway failure), at least when this movement involves a global displacement of the structure. The lateral displacement of an individual bar that buckles in a statically indeterminate structure may not be sufficient to lead to a global collapse. There are yet some cases, on the contrary, when a real failure has a particular fragile character and these are not so easily detected. The experience of the user is here a key factor.

Bernoulli Hypothesis The workhorse for modeling of building structure framing in fire is the Bernoulli beam finite element. It has yet to be understood that the hypothesis of plane sections remaining plane and perpendicular to the longitudinal axis has some consequences; some failure modes are not covered by this type of element. These are namely the shear failures, the slip between reinforcing bars or prestressing tendons and the concrete and the lack of rotational capacity due to local buckling. Any steel section, as thin as it might be, is seen by a Bernoulli beam finite element as a compact section with infinite rotational capacity. If local buckling is expected to be a crucial issue, it is still possible to rely on

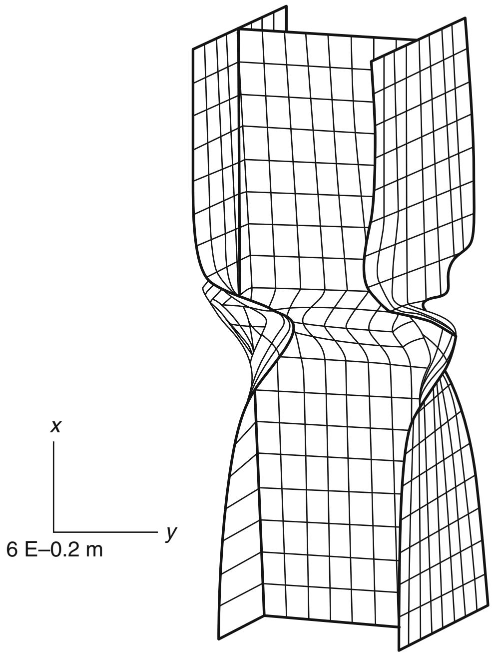


Fig. 52.21 Local buckling in a steel column

shell finite elements, see Fig. 52.21, but these are numerically more expensive elements and they will normally be used to analyze single members or subassemblies as opposed to complete structures.

Connections Connections between bar-type elements are most conveniently modeled on the basis of a simple hypothesis with respect to the relative rotation between the members: the connection is either assumed not to transmit any bending moment at all—hinged or pinned connection—or the relative rotation of the members is supposed to be null—rigid or fully fixed connection. In fact, the real behavior of any connection is between these two extreme situations. Any connection is semi rigid (or partially restrained), with some being more rigid than the others. The situation is more complex in the fire than at room temperature because of possible temperature differences between different components of the same connection and because of the presence of indirect effects of actions inducing axial forces (possibly different in sign between the heating and the cooling phase) and possible reversal in the sign of the applied bending moments. Very large displacements or thermal expansion can even totally modify the behavior of a connection. A connection that is very flexible at room temperature may become much stiffer in the fire if, for example, some gaps between different components are closed and these components enter into contact.

Significant research work has been done and is still being conducted on this topic with the aim of identifying and understanding the behavior of connections subjected to fire, especially of connections between steel, composite steel-concrete, and timber members. Valuable information has been derived, but this topic is still in the research phase and consideration of semi rigid connections in the fire situation is not yet common practice for real projects.

Limitations of Experimental Tests

Experimental tests also have their own limitations. Some are obvious, whereas others are not.

Cost is often mentioned as the first limitation. It is true that, in addition to the cost required by the fire laboratory for performing the test, other costs have to be added for the fabrication, transport, and disposal of the specimen, as well as for the time spent to define the test and assess its results.

The size of the structure is another limitation. Except under very exceptional circumstances, it is not possible to test a long span beam of, say, more than a few meters. Testing full-scale, complete structures is also seldom possible.

Time constraints may be another problem because there may be a significant amount of time between the day when the decision of a test is taken and the day when the results are available. Some time is required for buying the materials for the specimen if they are not available. The specimen has to be built and transported. Several months should be allowed for drying if concrete is involved. A time slot has to be found in the operations of a possibly busy fire lab, and the results have to be documented and interpreted. For some problems and questions that may appear during the erection phase of a building, time may simply not be available and experimental testing is, therefore, not an option.

Experimental testing has the same problem of boundary conditions as the analytical models but in the opposite direction. Pure and perfectly identifiable boundary conditions—such as perfect hinges and totally fixed supports for a mechanical analysis or an adiabatic condition for a thermal analysis—are easily considered in an analytical model, but they seldom represent the real boundary conditions that may exist in reality. On the contrary, it is very difficult to impose well-defined boundary conditions in an experimental test. Perfectly fixed supports would require an infinitely stiff testing machine. Some precautions that may prove technically complex and financially demanding have to be taken to approach a perfect hinge. It is very difficult to test a column under a perfectly centrically loaded condition. If, and this is often the case, the supports of the specimen are outside the furnace, the longitudinal heat loss along the member is not easily quantified, but may have a significant influence on the result.

The results of experimental testing have variability. Two identical specimens tested in the same laboratory will generally not yield exactly the same result. For example, variability has been observed and documented for concrete elements in which a slight difference in the concrete cover on the reinforcing bars or a difference in spalling may have a significant influence on the results. Variability may be higher in axially loaded columns than in elements with first-order bending moments because of the bigger relative influence of accidental eccentricities in the first case. Assemblies protected with membrane ceilings likewise can exhibit substantial fire performance differences attributable to the gypsum board material and its installation details, as can other assemblies or members that are more susceptible to physical integrity failures of the protection material or system. As a consequence, it is very difficult to make a parametric or sensitivity analysis by experimental testing, because the influence of the parameter that is analyzed may be hidden by the noise produced in the results by the variability linked to other factors. This is not the case, of course, if the number of experimental tests is statistically significant—and this may require a very significant number of tests—or if the influence of the analyzed parameter is overwhelming. Because of this variability, a so-called validation of any analytical model by comparison with the result of one or even a few experimental tests may be inconclusive.

Summary

This chapter reviews the key fundamentals of structural fire engineering and introduces the more advanced analytical methods (finite element–based for computers) that are emerging. It relies on the existing background derived from decades of prescriptive fire resistance testing and design and provides an advanced framework that enables solutions for performance-based objectives. Limit states structural design principles, fire loads and resistance, reliability, heat transfer, and the basics of structural analysis are presented. Several of the important variables affecting response to severe fire exposures are described, all of which can influence potential for major damage and collapse. These include thermal strains; local, member, and frame instability; floor slab effects and catenary action; connection stiffness and strength; nonuniform heating; material properties; and limit states.

The highlight of this chapter is contained in the insights and nuances of higher-order structural fire analysis, including the nonlinear elastoplastic regime, for different types of problems, exposures, primary loads, construction materials and elements, and levels of discretization. This information draws heavily from research, international sources, and engineering experience in the field. Modeling considerations for individual structural elements, substructures, and entire frames are given and further reinforced with a realistic overview of

their advantages, limitations, uncertainties, and possible issues with numerical solution convergence. These advanced analytical methods are also juxtaposed to the typical constraints and limitations of construction fire testing to create a balanced perspective on their capabilities for prediction of actual structural behavior.

Nomenclature

A_k	Structural fire effects, typically the construction material's strength and stiffness reductions caused by the fire's heating
D	Nominal design dead load
E	Youngs modulus of a material
f_y	Minimum specified yield strength
L	Nominal design live load (gravity)
L_{fire}	Design values of the load effects (direct effects resulting from the applied loads and indirect effects resulting from restrained thermal expansion) expected to be simultaneously acting during the fire event
M_n	Nominal flexural strength
M_u	Required (factored) flexural strength
P_n	Nominal axial strength
P_u	Required (factored) axial strength
x	Strong principal axis of the cross section
y	Weak principal axis of the cross section
S	Nominal design snow load
t	Time
T	Temperature
R_{fire}	Available structural resistance under the particular high temperature conditions, including the effects of degraded material properties

Greek Letters

$\Upsilon_{M,1}$	Partial safety factor for material strength
$\Delta\sigma$	Variation of mechanical stress
$\varepsilon_{mechanical}$	Mechanical strain
$\varepsilon_{thermal}$	Thermal strain
ε_{total}	Total strain
ϕ_c	Resistance factor for compression
ϕ_b	Resistance factor for flexure

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