

# Chapter 2

## The Fire-Resistive Principle



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### 2.1 Time Domain

Standard fire resistance design is based on the time domain. Standard fire testing serves as the basis of standard fire resistance design. Accordingly, fire resistance is defined as an hourly rating based on the results of standard fire testing.

Standard fire testing exhibits a mock-up fire-resistant assembly to a relatively intense fire exposure by means of furnace apparatus. This test method is predicated on the assumption that the test assembly is representative of actual field construction to a certain extent. However, the size limitation of furnaces restricts assembly sizes. For instance, floor assemblies are typically tested at spans no greater than 17 ft. (5.2 m), whereas an actual floor span may be much greater.

Each standard fire test uses the same temperature history that continually rises in temperature, to heat the test assembly with an established set of failure criteria [1]. Fire resistance directories provide a list of fire resistance-rated assemblies that

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have been qualified in accordance with standard fire testing [2]. These listings describe the process or details of construction that are commensurate with mock-up assemblies that have been qualified through testing. Within this framework, evaluation of structural fire protection reduces to the selection of qualified assemblies from available listings to meet prescribed levels of fire resistance.

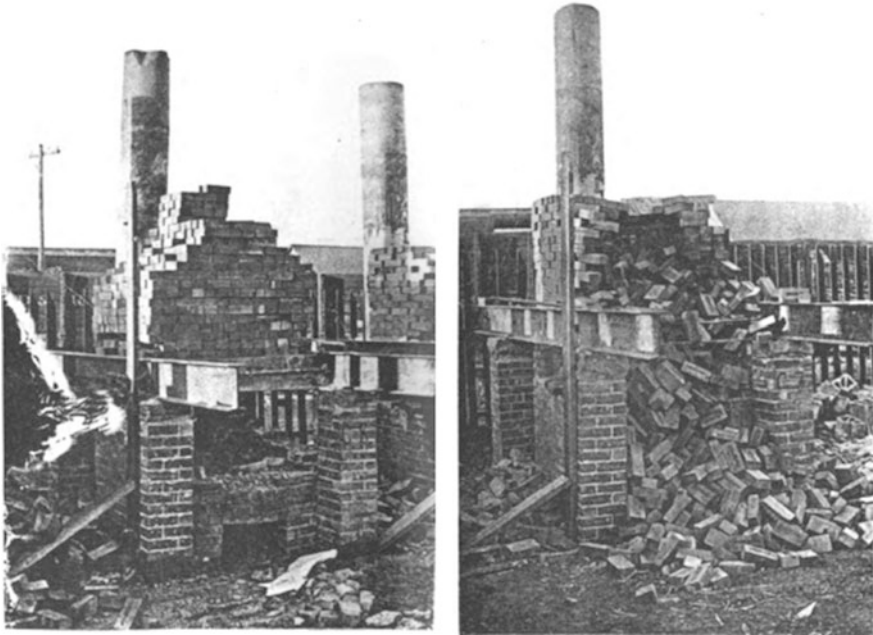
### ***2.1.1 Origins of Fire Resistance***

There exists a reference to the history of the subject of fire resistance evaluation [1]. Below is additional information that is meant to expand upon that resource. Digitization efforts in the last decade have made additional papers available in the public domain.

The origins of contemporary fire resistance began in the late nineteenth century in the aftermath of various city conflagrations, such as Chicago in 1872 and Boston in 1874. The outcome of these severe fires led to a surge in ad hoc fire tests of building elements. These tests primarily considered new reinforced concrete elements (beams and slabs). Between 1870 and 1890, the terminology called “fireproof” was adopted in practice. Fireproofing is strictly defined as incombustible construction [3]. These fireproof tests were qualitative in nature. Tests were often performed as a public demonstration of a building element constructed by a material manufacturer, supported on stilts, and burned using timber logs in random placement. They often were unloaded. Measurements (temperature and deflection) were often not recorded. Confidence in the building element was achieved by the nonappearance of “failure-collapse” with little science to validate manufacturer claims. Tests were published in newspapers as spectacles and there existed little scientific articling or reports that survive today.

In the 1890s, ad hoc testing was considered unacceptable through the eyes of architects when assigning competing assemblies for design that were claimed to be fireproof [4]. This led to the concept of fire resistance. From this point, testing considered quantitative performance—actual measurement and record keeping of the tests. Tests of building elements were compared using a more careful and rationalized test control method. Measurement of deformation of the building elements was made to define failure criteria—though collapse was often deemed being defined as failure.

One of the first “fire-designed” buildings using early principles of fire resistance was the Denver Equitable Building. The architects were faced with choosing three competing flooring (arch) systems made of terra-cotta, which were said to be fireproof [4]. The manufacturers of these competing flooring systems each argued that each of their products was superior. A demonstration-style test defined by the architectural firm Andrews, Jaques and Rantoul was organized for each flooring system. The test utilized the same target temperature of assault (gas temperatures of approximately 600 °C) and the flooring systems were ranked accordingly. Note that



**Fig. 2.1** Denver equitable building fire tests

these tests were extensively documented when performed in 1890 having a 17-photo set of loading and failure conditions; see Fig. 2.1.

In 1896 and 1897, two very different test series were organized by the New York Department of Buildings, led by researchers from the Mechanical and Mining Engineering Department at Columbia University. One test series utilized a controlled furnace for various building elements (led by Sylvanus Reed) [3], while the other utilized a testing procedure similar to the aforementioned Denver tests specifically for floors (led by Ira Woolson) [5].

The element testing by Reed established principles very similar to the ASTM standard fire that would follow in 1918 as well as some contemporary themes argued today for fire testing. Reed's tests relied on using a gas-fueled furnace to take advantage of the control of temperature. Reed documented various limitations for establishing test simplicity despite the broad objective of his test: *“steel or iron columns, girders, and beams, must be made on a full working scale and under the actual conditions, as far as possible, which would be obtained in a fire.”* Three different fire severities, based on occupancy type, were established as the metric for this series. The test parameters were defined accordingly by consultation from a committee. The objective was: *“To be a standard it must contemplate all fire possibilities, even the most remote, pertaining to those conditions . . . to establish a datum level from which allowable variations may be determined.”* The fire, he specified, would be run in a furnace as one of the three cases: (1) 1371 °C for 6 h—warehouses; (2) 648 °C for 1 h—commercial store; or (3) 371 °C for 30 min—office

building or house. All tests were under an applied service load. Temperatures were measured using a pyrometer. Reed justified that all buildings should be expected to resist a conflagration, as to quantify what expected damage state would occur. This was to inform the insurance industry. His tests were documented in the Journal of the Franklin Institute and are readily available to the interested reader [3].

At this time, engineers debated and attempted to influence the creation of these early tests. Abraham Himmelwright publically advocated that *“The object of all tests of building materials should be to determine facts and develop results that may be of practical value in future designing. In order that such facts and results may have real value, three conditions are necessary: first, that the materials tested shall be identical with what is commercially available in the open market; second, that the conditions, methods, and details of constructions conform exactly to those obtainable in practice; third, that the tests be conducted in a scientific manner.”* [6] He also stated that the design of structures had to resist thermal loading caused by fire: *“The actual and relative expansion of the materials due to heat and deflections caused by unequal heating must receive careful consideration . . . The limit of safety is in some cases dependent upon temperature and in other cases upon expansion.”*

Of note, both Reed and Woolson studied under Frederick Hutton of Columbia University. Hutton was an expert of furnace design [7]. While Woolson would start using wood-stocked furnaces, he would eventually advocate the use of a gas furnace by the inception of the standard fire test.

The tests performed by Reed were largely intended to be for informing the insurance industry and public. They were not meant as proprietary testing. Although this is not explicitly stated as the reason Reed’s tests ceased, the lack of funding may have contributed. It is interesting that Ira Woolson’s tests were more aligned to ranking proprietary systems where the material manufacturer often paid to test their systems. Ira Woolson’s tests would eventually form the basis of contemporary fire resistance as defined by qualification (standard) fire tests as per below.

Ira Woolson oversaw the second test series. His tests considered primarily flooring systems at first. The original test criterion called for a steady-state temperature of 1093 °C. The test temperature was defined in 1896 by the engineer Gus Henning, chief engineer of the New York Department of Buildings. Temperatures were reached by feeding a wood fire furnace which was beneath the loaded flooring assembly and the duration of heating was meant to be held for over 5 h (Fig. 2.2).

After Ira Woolson’s initial tests in 1896–1897, it was decided to specify a less severe fire exposure. The new test standard [8] called for a sustained “average” gas-phase temperature equivalent to 927 °C (1700 °F) for 4 h (with peaks still at 1093 °C (2000 °F)), hose stream cooling, and finally residual testing to higher loads (four times the sustained fire service load) for a further 24 h. If after this test the floor’s deflection did not exceed 1.4% of its span, the element was assumed to have “passed.” The test still used a wood-stocked furnace. The thermal scenario was intended to be more severe than a real fire. Woolson at the time advocated that *“no ordinary room would have enough inflammable material in it to maintain a 1700°F fire for more than 30 minutes.”* The basis for this heating regime was firefighters’ qualitative experience in New York. A complete catalogue of nearly 80 flooring

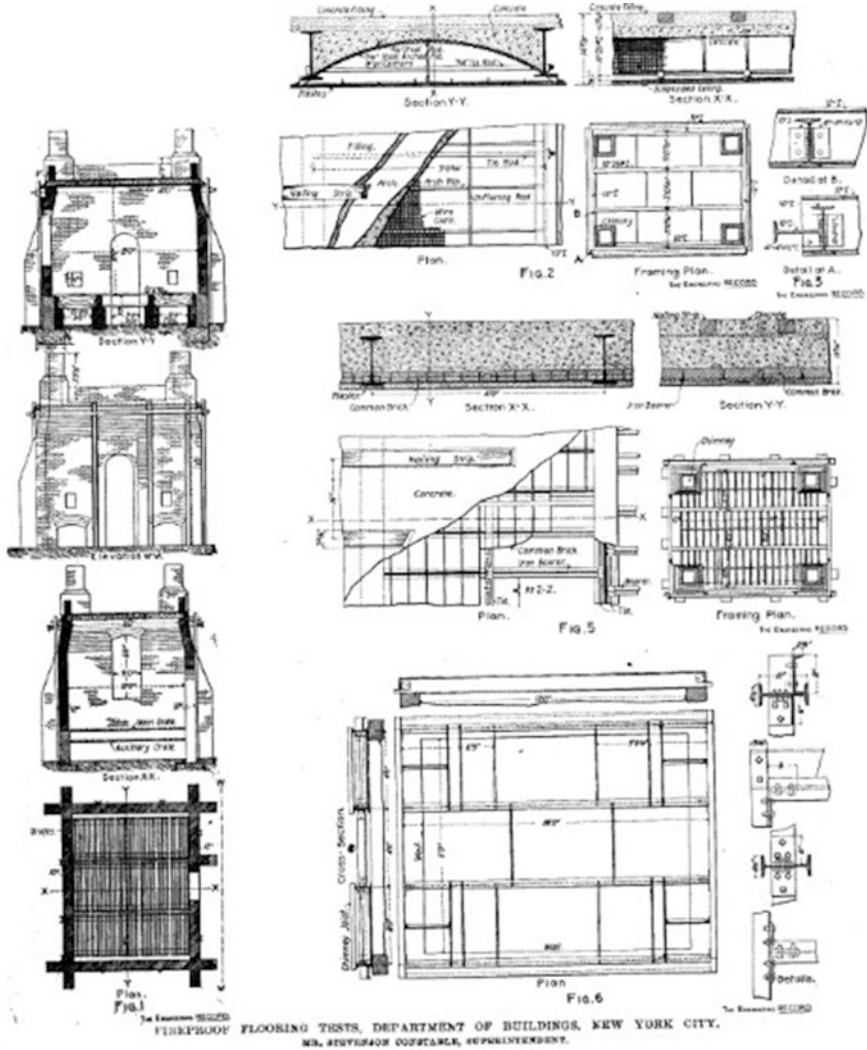


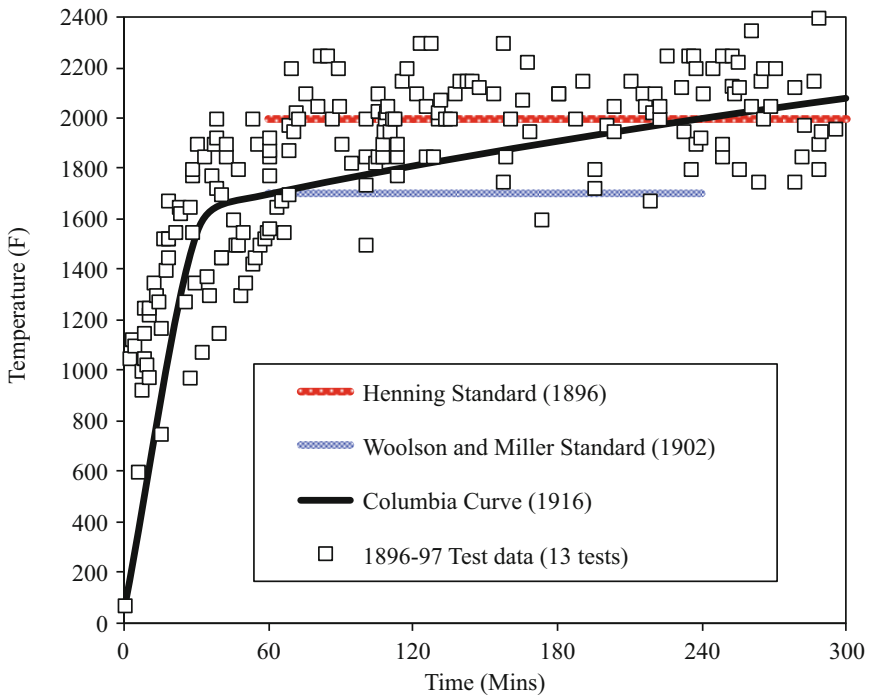
Fig. 2.2 1897 New York fire tests by the Department of Buildings [5]

systems was published by Woolson at the International Association for Testing Materials Conference in 1912 [9].

These early standard fire tests by Woolson were often criticized during this period of time [9–11]. Formerly the tests were not standardized at this time, and not widely adopted outside of New York. They were the subject of the Mazet Inquiry of 1899 which alleged corruption in the tests. These tests were followed by decreased influence of the city in the tests, and more control by research bodies to ensure their integrity. In response to the change in leadership of the test series, Gus Henning

in 1905 penned an open editorial in the New York Times where he publically criticized the current test procedure by Woolson: “*Other fakes I desire to call attention to are the fire tests now being made in New York City at temperatures of only 1,700 °F. I herewith wish to declare fire tests of materials made at average temperature of 1700°F as shams and frauds. They do in no sense of the word determine the fire proof qualities of materials.*” Henning’s reference to 1700 °F (927 °C—the 1-h mark used in the standard fire today) was in relation that real fires have more severity and that materials would behave differently under this severe heating. Following criticism towards the New York building structure fire test series, various construction material agencies lobbied for change [9]. This effort was mobilized by Ira Woolson at the American Society of Testing of Materials (ASTM). A new fire test standard evolved and was proposed in 1916. The intention was to shift away from floors and to consider columns.

There is no publically available documentation that explicitly defines the origins of the standard fire curve that was created in 1916 and actually still used today to assess fire resistance. Examination of data appears more of a compromise of the Henning 1897 and Woolson 1902 standard. Careful plotting of test data from the 1897 tests illustrates that the standard fire curve intercepts these points, as well as achieves a linear fit between 1 and 4 h of the Woolson curve adopted in 1902 and the Henning curve from 1897. This is plotted in Fig. 2.3 for the interested reader and requires continued research to definitively answer.



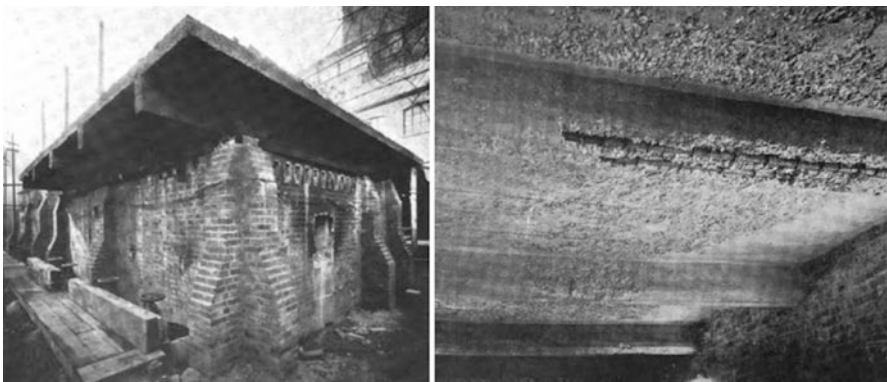
**Fig. 2.3** Evolution of fire time-temperature curves as adopted in the USA



The 1916 curve was used for the first time to test a series of timber, steel, and concrete columns. The criterion for the test was published by Simon Ingberg in the 1921 document: *Fire Tests of Building Columns* [12]. The test procedure used was very similar albeit with technological advances to the modern ASTM E119 fire test standard [1]. The tests in brevity considered using a controlled fire-time curve on loaded columns using gas-controlled furnaces. Gas furnaces could better control the time-temperature curve in linear fashions (except for timber which gave off its own heat). Even in the 1920s however, it was widely known that the standard fire was by no means indicative of a real fire. Ingberg reported that *“it is necessary to assume maximum probable conditions both with regard to building contents and air supply, as considered with respect to intensity and duration of a possible fire. Compensations and adjustments between intensity and duration may be necessary under some conditions in order to approximate a fire duration having intensity equivalent to that of the exposure of the fire test.”*

Efforts principally by Simon Ingberg [13] began to correlate a fire severity—using measurements from real burnout compartment tests—to the standard fire curve based on the “equal area concept.” Other researchers continued with the development of new concepts of equivalent fire severity based on other severity metrics (“maximum temperature concept,” “minimum load capacity concept,” and “time-equivalent formulae”). Buildings could then be reclassified, not only by fire activation risk, but also by functions of fuel load, and “equivalent” standard fire resistance times could then be specified for building elements.

Changes to the standard time-temperature curve were made through the years in various iterations of ASTM standards (though with increasingly less emphasis on residual capacity of the elements after a fire and to exploit technological advances for test control). The fire community has largely followed the original testing procedure for construction materials and elements under fire. Careful examination of literature will show similar initiatives that were underway in Europe (see Fig. 2.4); however, the momentum for developing standardized fire testing would not appear until BS 476 was adopted, which largely mirrored the ASTM fire standard. BS 476 would



**Fig. 2.4** Reinforced concrete floor tested by Edwin Sachs, 1906 [14]

later evolve into ISO 834. Further information on European background may be found elsewhere [15].

By the early 1980s, overreliance on standard fire testing was widely recognized as limiting innovation in architecture and construction, and technical papers began to appear which openly questioned the applicability of these tests [8]. Fire engineering researcher David Jeanes commented in 1982 [16]: *“although the traditional approach of assigning time for a given structural element or assembly allowed for a comparative measure between different types of construction; it is hard pressed to represent actual structural performance in a real fire due factors of restraint, redistribution of loads, moment resistance, as these are difficult to quantify and duplicate in tests.”* The standards today recognize fire resistance as the time duration that a “mock-up” assembly is able to withstand furnace heating based upon standard fire testing requirements and acceptance criterion defining test end.

### 2.1.2 Qualification Testing

Building elements can be tested under controlled conditions in a standard fire test, also known as qualification tests. Standardized tests are jurisdiction dependent and include multiple similarities as they typically originate from the ASTM standard. To date, the more popular standards referenced are ASTM E119 and ISO 834. The goal of qualification testing is to determine a time-based fire resistance rating. The test is intended to allow comparison between various assemblies used nationally. This test uses the standard fire curve (time-temperature curve) which continually rises in temperature with time. In the testing furnace, burners are controlled in order for the temperature inside the furnace to follow the designed time-temperature curve. Table 2.1 illustrates the current specified control temperatures of the test.

Control of temperatures can be very accurately defined in modern furnaces and has strict tolerances as defined in the standards. Tests typically are conducted for

**Table 2.1** Time and temperature values for ASTM E119 and ISO 834

Time (min)	ASTM E119 temperature (°C)	ISO 834 temperature (°C)
0	20	20
5	538	576
10	704	678
30	843	842
60	927	945
120	1010	1049
240	1093	1153
480	1260	1257



walls, floors, beams, columns, penetrations, and junctions. The test is intended to consider as-built construction; the assembly is placed in a rigid frame and positioned inside, next to, or over the standard testing furnace depending on the member type. Once placed a likely service load is applied. The load is maintained as the standard time-temperature curve is applied. The test is continued until a failure criterion is reached (Sect. 2.1.3).

Standard fire testing does not account for physical parameters governing fire behavior such as fuel load or ventilation. Moreover, furnace size limitations impact the size of assemblies that can be tested, as well as impact the restraints and loads acting on a test specimen. Therefore, standard fire testing is not representative of conditions in real fires. This is stated in a disclaimer in ASTM E 119 as “*this standard is used to measure and describe the response of materials, products, or assemblies to heat and flame under controlled conditions, but does not by itself incorporate all factors required for fire hazard or fire risk assessment of the materials, products or assemblies under actual fire conditions.*” [1] However, the principles of the test form the basis of many global jurisdictions’ hourly fire ratings given to various infrastructure. The reasoning is defined as above through the contextual history of the test.

There are certain limitations of assembly qualifications that the practitioner should consider, and these largely deal with the realism of the test. Many practitioners have subsequently advocated the use of consistently crude approaches for structural fire testing. In the case of qualification testing, it is largely appropriate for building elements only. When taken out of this context, there are limitations in its interpretation to reality (method of construction, appropriate element size, loading configuration, thermal boundary, cooling, etc.). Lastly, important consideration should be made regarding reproducibility. While advances in furnace control have been made through the avocation of plate thermometers, no one furnace is strictly the same from laboratory to laboratory.

### 2.1.3 Testing Criteria

Deflection, specimen temperatures, and sometimes strains are monitored during standard fire testing. The performance of an assembly is measured as the period of resistance to a standard fire before failure (Fig. 2.5). Failure criterion is denoted as either stability, integrity, or insulation. For all assemblies, some criteria are not applicable (see Table 2.2). Stability references that an assembly should not collapse (or in some cases limited to how much or how fast deflection can be). Notably, the load-bearing capacity of columns in the latter can be defined by a limiting axial contraction of  $h/100$  (mm), and a limiting rate of axial contraction of  $3 h/1000$  (mm/min). ASTM E119 does not mention a limiting value for axial contraction. However, deflection criteria for beams under both standards mentioned are given as maximum total deflection of  $L^2/400d$  (mm or in) and maximum deflection rate per minute of  $L^2/9000d$  (mm/min or in/min) after the maximum total deflection has been



**Fig. 2.5** Conclusion of a standard fire test of a wooden floor (photo by Gales)

**Table 2.2** Typical failure criteria by assembly type

	Stability	Integrity	Insulation
Partition/door		x	x
Floor/ceiling/wall	x	x	x
Beam/column	x		

exceeded. Deflection criteria (as defined above) derive from concrete and steel tests performed in the 1950s [17]; they are generally defined as limitations to prevent failure of the slab into the furnace that can cause significant damage. Integrity references to the ability of the specimen to not allow the passage of flame. The mechanism used to assess this is typically the ignition of cotton waste on the unexposed surface. Lastly insulation is the ability of the assembly to not allow excessive heat transfer. Depending on the standard this is considered less than 180 °C.

### 2.1.4 “Restrained vs. Unrestrained”

For certain fire resistance listings, the designer must judge whether a “restrained” or “unrestrained” classification is appropriate for the application. It is common for architects to task fire protection engineers or structural engineers to make such judgments. Many listings permit less applied fire protection to achieve a certain fire resistance rating if a “restrained” classification is adopted, as compared to an “unrestrained” classification.

ASTM first introduced the “restrained vs. unrestrained” concept in the 1970s [18], based on the notion that thermal restraint provided by the furnace enclosure generally enhances the performance of fire-resistant assemblies. For instance, the thrust forces generated by restrained thermal expansion can help to reduce the

deflection of steel beams under heating. Since beam deflection is an acceptance criterion of the test method, this effect is usually beneficial as it pertains to fire resistance. An assembly is considered “restrained” if it bears directly against the edges of the furnace at the outset of the test [19]. For reference, the UL Fire Resistance Directory states that the furnace enclosure boundaries provide approximately 850,000 kip-in. of flexural stiffness [20], which is significantly higher than that provided in situ in most cases.

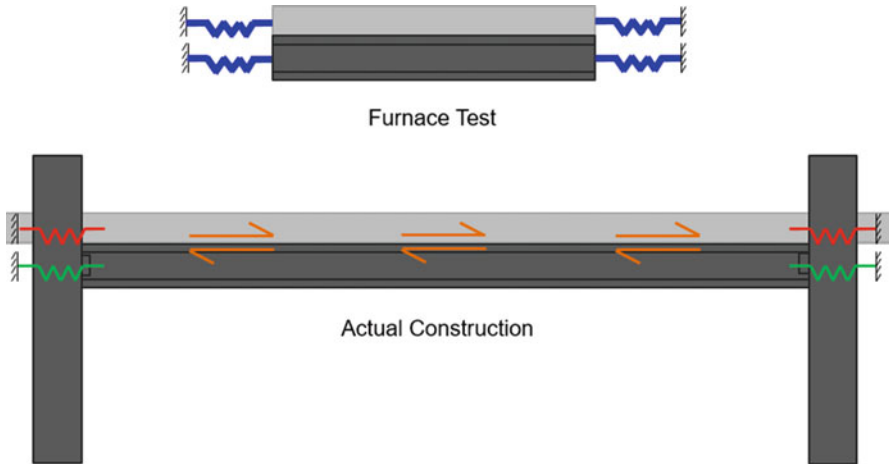
The UL Directory [20], the American Institute of Steel Construction (AISC) 360 standard [21], and certain publications provide guidance on determining restraint conditions, but the classification of an assembly as either “restrained” or “unrestrained” is ultimately governed by the judgment of the designer. Since standard furnace testing does not consider structural system response, but only the response of components, this judgment is often inconsistent among designers.

The effect of thermal restraint must be carefully evaluated since it may dominate the behavior of a structural system under fire exposure [22]. Complicating matters, it is known that a multitude of factors influence restraint conditions (e.g., varying spring stiffness), and these factors may *increase or decrease* structural system endurance under fire exposure. For instance, thermal restraint may generate forces sufficient to cause yielding or fracture of connections (as illustrated in Fig. 2.6), perhaps precipitating structural collapse. Alternatively, thermal restraint may limit the deflection of structural members and provide added stability.

The conditions of restraint differ between standard fire testing and in situ conditions. In actual building construction, restraint of structural assemblies occurs when the surrounding structural system resists their thermal expansion when exposed to



**Fig. 2.6** Steel beam flange buckling



**Fig. 2.7** Thermal restraint (furnace test vs. actual construction) [23]

heating. During standard fire testing, restraint is provided by the furnace enclosure. Furthermore, a steel beam and concrete slab would be restrained equally during a furnace test. In actual building construction, the beam would typically have less resistance to thermal expansion as compared to the slab, resulting in differential longitudinal movement under fire exposure (as illustrated in Fig. 2.7). Due to the differences between test conditions and actual construction, there continues to be ongoing confusion and debate concerning this concept.

Several organizations have conducted furnace testing to better understand the influence of restraint on the level of fire resistance achieved. AISC funded furnace testing of steel floor assemblies, which found that restraint of the furnace frame provided no fire resistance benefit [24]. The National Institute of Standards and Technology (NIST) performed furnace testing of steel trusses (Fig. 2.8) and found that an unrestrained assembly achieved a higher fire resistance when compared to an equivalent restrained assembly [25]. These test results demonstrate that the effect of restraint varies among different structural systems.

The ASCE/SEI 7 standard [26] provides guidance on how designers should consider thermal restraint generally. Specifically, ASCE/SEI 7 Section E.2 states that thermal restraint is entirely dependent on adjacent structural framing and connection details, which are not contemplated in standard fire resistance design. Accordingly, Section CE.2 states that furnace testing does not provide the information needed to predict the actual performance of a structural system under fire exposure, since furnace testing qualifies assemblies in isolation without interconnectivity or interaction with the surrounding structural system.

When structural fire engineering is employed, analysis of structural system response inherently considers the amount of structural restraint actually present. However, when standard fire resistance design is used, the degree of restraint is left to the judgment of the designer, for building codes (e.g., IBC) do not provide specific



**Fig. 2.8** NIST testing of steel trusses [25]

prescriptive classification. Hence, the designer is forced to make a somewhat conflicted judgment.

Recently, the industry has begun to reexamine the “restrained vs. unrestrained” concept in order to better serve designers going forward [23]. Many designers believe that clarification/reform of the “restrained vs. unrestrained” concept is needed. For instance, an industry consensus that is clearly stated in the IBC would relieve designers of the obligation to make uncertain judgments within standard fire resistance design. Such judgments are better reserved when employing structural fire engineering.

Until an industry consensus is included in building codes, designers may choose to take a conservative approach when classifying restraint conditions within standard fire resistance design [27]. Notably, IBC Section C703.2.3 states that in situ conditions should be considered unrestrained unless structural documentation is provided that demonstrates a restrained condition in actual construction [28]. In all cases, the authority having jurisdiction may be consulted as to the proper interpretation for a given project.

### ***2.1.5 Empirical Calculation Methods***

Assemblies that are qualified through standard fire testing are published in directories such as the UL Fire Resistance Directories [20], which contain hourly ratings for beams, floors, roofs, columns, walls, and partitions. These listings have very specific construction requirements that are commensurate with the test mock-up.

Even though fire resistance directories are quite lengthy, the ability of designers to achieve project goals is routinely inhibited by this empirical framework, especially when unique or nonconventional architecture is proposed.

Standard fire tests can be very costly to conduct (e.g., US\$100,000 to administer a single test). When members or assemblies and their passive fire protection are similar to those already tested, industry-accepted empirical calculation methods may be employed to determine the fire resistance rating. Generally, these empirical calculation methods only interpolate between established test data, for extrapolation would not be proper given the empirical nature of standard fire testing.

Chapter 7 of the International Building Code [29] is a common reference for industry-accepted empirical calculation methods. This chapter includes table lookups and equations to determine the fire resistance of generic construction materials that have been thoroughly tested. For instance, independent of a fire resistance directory, it can be derived via a table lookup that a 5 in. solid wall thickness of siliceous aggregate concrete provides 2 h of fire resistance. Similar to Chapter 7 of the IBC, ASCE/SEI/SFPE 29 [30] is a commonly used standard that exclusively provides empirical equations for use in standard fire resistance design.

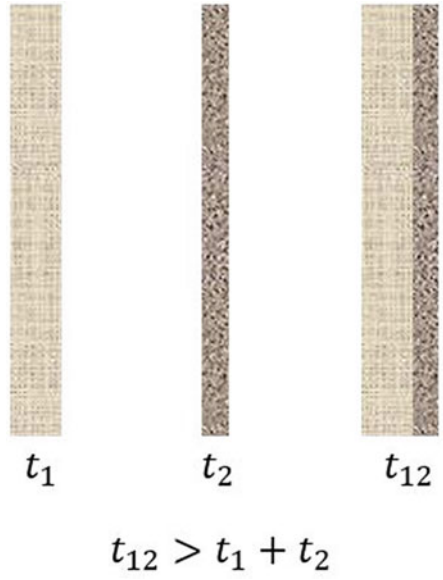
### ***2.1.6 Equivalence Methods***

Equivalence is a term used within standard fire resistance design, which can be defined as having equal or better fire resistance as compared to a tested assembly. The evaluation of equivalence must be conducted within the context of the standard fire test methodology and its specific acceptance criteria, and not be linked to actual structural systems and actual fire exposures. In other words, for a proposed fire-resistant assembly to be deemed as equivalent to a previously qualified assembly, it must be demonstrated that such an assembly would perform equally or better if exhibited to a hypothetical standard fire test. Equivalence may be demonstrated using either qualitative or quantitative approaches.

Qualitative approaches harness relatively simple logic to contemplate the anticipated performance of a given assembly as compared to another. Most notably, the “Rules of Harmathy” can be used to perform a quick assessment of fire resistance when fire test data are not available [31]. Harmathy’s Rules that are pertinent to standard fire resistance design include the following:

- Parallel insulating layers perform equal to or better than the sum of each individual layer tested separately (Fig. 2.9).
- Adding insulating layers does not decrease the fire resistance.
- Adding an air gap within parallel insulating layers does not decrease the fire resistance (Fig. 2.10).
- The further an air gap is located from the exposed surface, the higher the fire resistance.
- Increasing the width of an air gap has negligible effect on fire resistance.

**Fig. 2.9** Harmathy's rules  
(parallel layers)

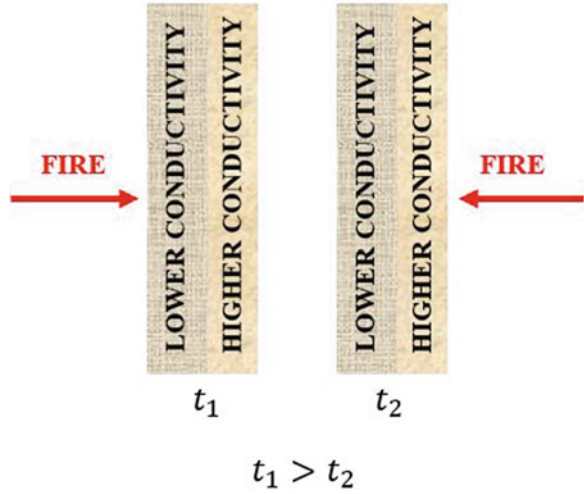


**Fig. 2.10** Harmathy's rules  
(air gaps)

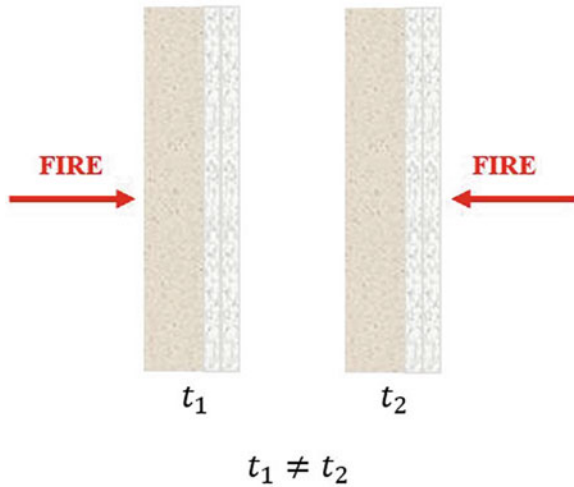




**Fig. 2.11** Harmathy’s rules  
(layer thermal conductivity)



**Fig. 2.12** Harmathy’s rules  
(side of fire exposure)



- Locating insulating layers with a lower thermal conductivity towards the exposed side provides higher fire resistance with all else equal (Fig. 2.11).
- For a given set of insulating layers, the fire resistance achieved for fire exposure on one side does not necessarily equal that if the fire exposure is from the other side (Fig. 2.12).
- If the construction is not susceptible to explosive spalling, the presence of moisture in the insulating layers increases the fire resistance.

Quantitative approaches seek to explicitly simulate the performance of a test specimen during a standard fire test. In this case, the designer would represent the

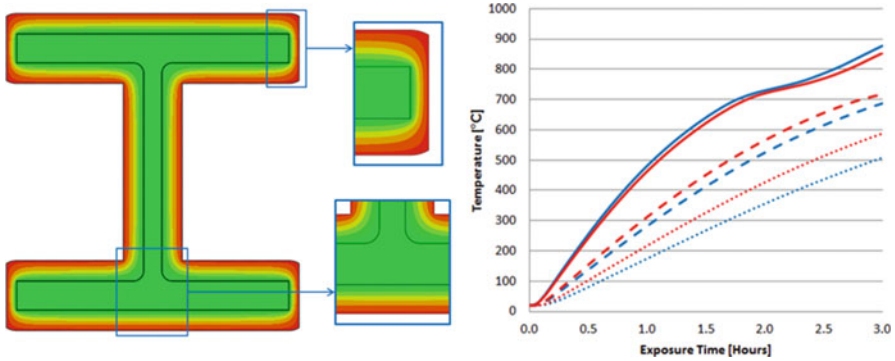


Fig. 2.13 Thermal model of an unloaded column test

span length characteristic of a furnace (e.g., 17 ft.) and expose the specimen to standard furnace heating. Also, the fire resistance of the specimen would be qualified per the acceptance criteria. This type of approach usually requires the use of finite element modeling that has been validated against similar furnace testing results. For instance, Fig. 2.13 shows the results of a 2D thermal model used to understand the heating of a protected steel column during an unloaded column test. In this example, the effect of reduced insulation thickness at the column flange tips is studied.








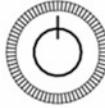
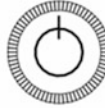
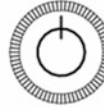
## 2.2 Strength Domain

Structural fire engineering is based on the strength domain. Principles of structural engineering serve as the basis of structural fire engineering (see Sect. 2.2.1). Accordingly, structural fire engineering explicitly evaluates all aspects of demand and capacity of structural systems under fire exposure. Within this framework, the demand on a structural system under fire exposure can be reduced by means of rationally allocated structural insulation (e.g., applied protective insulation), control of fuel loads, and/or other fire exposure mitigation techniques. Also, the capacity of a structural system to endure fire exposure can be increased by means of specific member sizing, connection detailing, and/or other measures to enhance structural robustness [32].

Figure 2.14 illustrates the difference in controllable design variables between standard fire resistance design (time domain) and structural fire engineering (strength domain).

### 2.2.1 Structural Engineering

Structural design in the USA primarily uses the load and resistance factor design (LRFD) method. This method employs a statistical based approach for predicting

	INSULATION	FUEL LOAD CONTROL	STRUCTURAL DESIGN	STRUCTURAL DETAILING	OTHER MITIGATION
STANDARD FIRE RESISTANCE DESIGN					
STRUCTURAL FIRE ENGINEERING DESIGN					

**Fig. 2.14** Controllable design variables [27]

loads and material strengths. Within this paradigm, a load effect is defined as the force in a member or an element (axial force, shear force, bending moment, or torque) due to the nominal load (e.g., self-weight, snow weight, wind pressure, seismic inertia). Each member and element has specific structural capacities (e.g., flexural capacity) to withstand load effects. A limit state is reached when a specific capacity or capacities of a member or element no longer fulfill the relevant design criteria (e.g., flexural yielding). Accordingly, to qualify the safety of a conventional design, structural engineers must calculate the demand-to-capacity ratio (DCR) for each applicable limit state. The selection of applicable limit states is based in part on a designer's ability to identify all conceivable modes of failure or mechanisms in which the structural system could conceivably fail.

LRFD criteria reduce the probability of the load effect exceeding a capacity to an acceptable level. Accordingly, this method results in members and elements that are sized to withstand all considered load effects during the design life of the structural system, with an appropriate level of reliability for each relevant limit state. For instance, steel beams designed to withstand gravity loads have a probability of exceeding their flexural limit state on the order of 0.005 to 0.0005 on a 50-year basis, corresponding to reliability indices of approximately 2.5–3.3 [33]. Specific limit states that have a higher consequence of failure typically have higher target reliability. For instance, brittle failure modes, such as concrete column crushing, occur with little warning and usually inhibit load redistribution.

LRFD results in a more consistent degree of reliability across different design scenarios, as compared to preceding deterministic approaches, such as allowable stress design (ASD). For instance, consider two roof structures: a reinforced concrete beam/slab and a reinforced concrete plate, designed for the same snow load using the same allowable stresses per ASD. The first structure has considerably higher dead load as compared to the second. Since the dead load can be estimated with much more precision than the snow load, the roof having the high ratio of dead to snow load would have a lower probability of failure than the lighter structure. Accordingly, LRFD accounts for the variability of individual loads through load effect

factors. In Europe, the Eurocodes adopt a similar approach by utilizing concepts of limit states and associated design factors. Also, the ISO 13824 standard, which is adopted by a number of countries across the world, provides guidance for risk-informed design of structural systems [34].

The determination of structural load effects in the USA is primarily conducted in accordance with the ASCE/SEI 7 standard [26]. This standard requires that structural members and elements be designed considering certain load combinations, which comprise individual load effects multiplied by specific load factors. The load combinations used for conventional structural engineering design pertain to those resulting from dead (i.e., self-weight), live (i.e., movable weights), snow, rain, wind, and seismic nominal load effects. Nominal loads are frequently defined with reference to a probability level (e.g., 50-year mean recurrence interval wind speed).

The nominal dead, live, and snow loads provided in ASCE/SEI 7 are based on a combination of measured data and engineering judgment. Thus, there is a small but finite probability that a nominal load per this design method will be exceeded in a given year. For instance, stochastic models of typical building operations were used to develop nominal live loads based on the occupancy or use. Based on the principles of mechanics, the nominal loads produce load effects that are used in the load combinations. If the relation between the nominal load and the resulting load effect is linear (which is typically the case in conventional design), the designer may apply the load effect factor to the nominal load for convenience when performing a structural analysis. The sum of a load combination produces the demand for DCR calculations.

In the USA, the determination of structural capacity is primarily conducted in accordance with standards produced by material organizations (e.g., AISC 360 [35]). Material organizations specify strength reduction factors that are typically less than unity, which are applied to nominal strength parameters used in structural calculations (e.g., yield strength, modulus of rupture). These factors are based on uncertainties associated with the strength of members and elements (e.g., material composition, dimensional tolerances) and the consequence of the failure limit state (e.g., concrete crushing). Also, strength reduction factors are intentionally set lower for structural connections as compared to structural members, reflecting the higher consequence of connection failures. For a given limit state, multiplying the strength reduction factor by the nominal strength parameter produces the capacity in DCR calculations.

Since independent material organizations govern the composition of corresponding material standards (and the load effect factors in ASCE/SEI 7 are constant irrespective of the material of construction used), the underlying reliability of structural designs varies across different building materials (e.g., steel, concrete, wood, masonry). Notwithstanding, the average reliability index  $[\beta]$  of a structural design using ASCE/SEI 7 load effect combinations involving dead, live, and snow loads is approximately 3.0. For wind and seismic loads, the average reliability index is approximately 2.5 and 1.8, respectively [36].

## 2.2.2 *Relevant Standards*

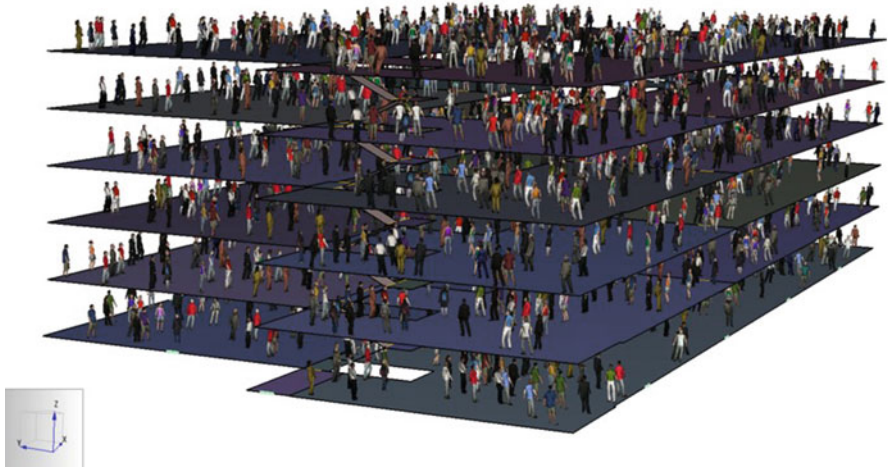
Since standard fire resistance design does not contemplate structural system performance or explicit performance objectives, there exists no practical method for a designer to quantitatively compare the level of safety provided by a structural fire engineering design to that provided by a standard fire resistance design. Furthermore, it is not reasonable to require that a structural fire engineering design be “equivalent” to a standard fire resistance design, which does not provide any affirmative quantification of structural fire safety. Hence, the industry consensus embodied in industry standards is absolutely essential for successful implementation of structural fire engineering [37].

The use of structural fire engineering constitutes an alternative methodology to meet project design objectives, as permitted by the alternative materials, design, and methods of construction provision in building codes, such as the IBC. Acceptance of structural fire engineering designs is elective and subject to approval by the authority having jurisdiction.

The SFPE Engineering Guide to Performance-Based Fire Protection [38] outlines the process for using a performance-based approach for building fire safety, which is applicable to structural fire engineering design. Notably, this standard provides guidance for creating a design brief, which serves as a memorialized agreement of the assumptions, performance expectations, etc. Also, this standard describes the role of various stakeholders. Unlike standard fire resistance design in which the architect typically serves as the responsible party for satisfying code requirements for structural fire resistance, structural fire engineering usually requires a team consisting of structural engineers, fire protection engineers, and possibly other design professionals.

In addition to designers, the involvement of the owner, building authority, and possibly peer reviewers would need to be addressed when employing structural fire engineering. If required, a third-party peer review is conducted independently by persons with appropriate expertise and experience to evaluate compliance of the proposed design with industry standards (e.g., ASCE/SEI 7-16), and any other requirements of the building official. The SFPE Guideline for Peer Review in the Fire Protection Design Process provides guidance on the peer review process [39].

ASCE/SEI 7 serves as the parent standard for structural engineering in the IBC. The current edition of this standard includes a new Chap. 1 section (Fire Resistance). In addition to being the first time that fire resistance has ever been addressed in this standard, this section commences a new industry consensus standard of care for structural fire protection practice in the USA, and other adopting jurisdictions [40]. The default option is for the designer to strictly adhere to the requirements and restrictions of standard fire resistance design per the applicable building code. The only permitted alternative to standard fire resistance design is structural fire engineering, as constituted in the standard’s new Appendix E section (Performance-Based Design Procedures for Fire Effects on Structures). Notably, the inclusion of



**Fig. 2.15** Occupant evacuation simulation

Appendix E in ASCE/SEI 7 marks the first time that fire effects are considered as an explicit design load in a US structural engineering standard [32].

ASCE/SEI 7-16 Appendix E is organized into six sections with associated commentary. Notably, Section E.4 specifies mandatory and discretionary performance objectives for structural systems under fire exposure. The mandatory performance objectives uphold the intended functionality of occupant egress systems. In all cases, the designer must explicitly demonstrate that the structural system would allow for a safe and complete evacuation of building occupants to a public right of way (e.g., roadway) in the event of an uncontrolled fire. This necessitates an ASET (Available Safe Egress Time) vs. RSET (Required Safe Egress Time) analysis, in which the determination of RSET involves consideration of occupant egress times (Fig. 2.15). The SFPE Guide to Human Behavior in Fire provides guidance on how to calculate RSET [41].

In addition to occupant evacuation, the designer must demonstrate that structural elements that support building refuge areas within the building (e.g., refuge floors) would remain stable during and after an uncontrolled fire event. Beyond the mandatory performance objectives, all other relevant performance objectives are classified as discretionary within ASCE/SEI 7-16.

Per ASCE/SEI 7-16, discretionary performance objectives may address issues such as tolerable levels of structural damage, structural support of ingress routes for firefighters, structural support of fire resistance-rated assemblies, and others. For instance, ASCE/SEI 7-16 Section CE.4.2 recommends tolerable levels of structural damage under fire exposure based upon the building's Risk Category assuming that all of the mandatory performance objectives are satisfied. For buildings that meet Risk Category I criteria (e.g., storage buildings), structural collapse from fire exposure is permissible if the collapse does not damage surrounding properties, including

buildings and infrastructure systems. For the majority of buildings which can be classified as either Risk Category II or III, the primary structural system (i.e., columns, structural elements having direct connections to columns, and lateral bracing elements) should remain stable under fire exposure and subsequent cooling. As such, damage to structural elements or assemblies that does not compromise the stability of the primary structural system or continuity of the load path is permissible. For buildings which are classified as Risk Category IV (e.g., hospitals), the entire structural system (including secondary structural elements) should remain stable under fire exposure and subsequent cooling which would allow for rapid reoccupation of areas not directly affected by fire exposure.

The need for and the scope of discretionary performance objectives must be agreed upon by project stakeholders, and this agreement should be memorialized within a Design Brief document. Even if discretionary performance objectives are not explicitly analyzed for a given project, fulfilment of the mandatory performance objectives may enhance structural performance in these respects (e.g., added structural robustness of stairways used by firefighters).

ASCE/SEI 7-16 Appendix E Section E.5 provides requirements for evaluating the heating of structural systems under fire exposure with reference to the NFPA 557 [42], SFPE S.01 [43], and SFPE S.02 [44] standards. The NFPA 557 standard establishes a basis for estimating building fuel loads. The SFPE S.01 standard provides requirements for evaluating thermal boundary conditions from fire exposure. Lastly, the SFPE S.02 standard provides requirements for heat transfer calculations based on the thermal boundary conditions. ASCE/SEI 7-16 requires that “structural design fires” be analyzed, defined as those that are uncontrolled by active measures, such as automatic fire sprinklers or firefighting activities.

The NFPA 557 standard provides a reliability-based method for calculating either localized or distributed fuel loads for use in fire exposure calculations. For enclosure-type fire exposures, the design distributed fuel load would be applicable. Unless the distributed fuel loads contained in a particular building are explicitly surveyed, the occupancy-based method in this standard should be used. The occupancy-based method involves calculating a fuel load risk factor that reflects the likelihood of an uncontrolled fire occurring based on National Fire Incident Reporting System (NFIRS) data, and a target  $\beta$ -value of approximately 5.0. The distributed fuel load risk factor is a function of the following:

- Occupancy type (e.g., educational).
- Construction characteristics (e.g., protected noncombustible).
- Presence or absence of active fire protection systems.
- Level of inherent and applied fire protection present.

Based on specific fuel load surveys and studies vetted by the NFPA, this standard specifies average and standard deviation values of the distributed fuel load for different occupancy types. These values reflect a 99% upper confidence bound. The design distributed fuel load is calculated as a function of these statistical values and the fuel load risk factor.



The Eurocode provides a similar framework for determination of fuel load density, which is a function of the nominal fuel load and specific risk factors per Annex E [45]. Notably, there is a risk factor that accounts for the presence or absence of active fire protection measures (including manual suppression). The Eurocode treats the nominal fuel load as a variable parameter with a Gumbel distribution, and suggests the use of an 80% upper confidence interval. These risk factors (as stated in the Eurocode background documents) were determined considering the  $\beta$ -value to be approximately 5.0 for a building design life of 55 years. Notably, some countries of Europe (e.g., UK) have not adopted the risk factors as presented in the Eurocode (with the exception of the reduction factor for the presence or absence of a fire sprinkler system).

The survey-based method in NFPA 557 serves as an alternative approach and involves the manual accounting of anticipated fuel masses and their respective combustion properties. Generally, a survey-based approach would only be used in cases where justification of a lower fuel load density (as compared to that calculated using the occupancy-based method) is warranted by special circumstances. Otherwise, there would be limited justification for the effort involved.

Based upon the fuel load, ignition(s), and arrangement of compartments and ventilation openings, structural design fires are often characterized as either an enclosure fire or a localized fire. Accordingly, the SFPE S.01 standard provides methods to determine time-dependent thermal boundary conditions on a structural system due to either an enclosure or a localized fire. Similarly, the Eurocode Annex A provides equations for parametric fire curves. However, the Eurocode allows for the use of the standard fire curve (used for furnace testing), which is unlike ASCE/SEI 7. Exterior fire exposures and traveling fires are also discussed for which design guidance is comparatively less robust. In special circumstances, it may be necessary to perform fire modeling which should be substantiated according to the SFPE G.06 standard [46].

The NFPA 557 standard specifies the extent of a structural design fire as either the entire building or that portion of the building that is bounded by exterior walls and/or by fire-rated boundaries that are capable of containing a fire for the entire duration through burnout. If a given floor of a building has no fire-rated boundaries, the entire extent of the floor must be assumed to be involved in fire. The SFPE S.01 standard currently specifies the extent of fire exposure similar to NFPA 557. Neither NFPA 557 nor SFPE S.01 currently provides specific guidance on the number of building stories that should be considered as involved in fire.

Based upon the time-dependent thermal boundary conditions from fire exposure, the thermal response of a structural system can be determined based upon the principles of heat transfer in accordance with the SFPE S.02 standard. Additionally, key sources for temperature-dependent thermal properties of conventional construction materials are identified in ASCE/SEI 7-16 Section E.5.

Based on the results of thermal response calculations, ASCE/SEI 7-16 Section E.6 provides requirements for subsequent structural response calculations. This section requires consideration of all heated members of a structural system, and those unheated members that induce thermal restraint forces. Additionally, the effect

of material strength and stiffness degradation must be considered, which may result in high deflections and deformations. Specifically for steel structures, AISC 360 Appendix 4 (Structural Design for Fire Conditions) provides added relevant guidance for designers [21]. However, AISC 360 Appendix 4 should not be relied upon exclusively for structural fire engineering designs since it lacks critical over-arching and material-neutral requirements. Notably, structural analysis scope and other baseline requirements are left undefined/open-ended. In such cases, ASCE/SEI 7-16 Appendix E requirements would govern.

Unlike standard fire resistance design, ASCE/SEI 7-16 Section E.6 requires that structural analyses include portions of the structural system that are subject to fire effects with consideration of unheated portions of the structural system that provide thermal restraint (see Sect. 2.1.4). A single-member analysis is permitted in cases where only a single member is affected by a fire without consequential effects from surrounding members. Otherwise, a systems approach that evaluates thermal expansion of heated sections and restraint by cooler adjacent framing is necessary. Additionally, analyses of structural system response to fire exposure must always consider the performance of structural connections.

Unlike conventional structural engineering design, ASCE/SEI Section E.6 allows for consideration of alternative sources of load-carrying capacity and load paths that are capable of being maintained following structural damage or degradation due to fire exposure (e.g., catenary action). Moreover, Section E.6 includes discussion of specific design considerations and critical failure modes for columns, floor systems, connections, and other structural components. Lastly, this section discusses proper transfer of results from heat transfer analyses to subsequent structural analyses, and key sources for temperature-dependent mechanical properties.

ASCE/SEI 7-16 Section E.6 requires the use of load combinations for extraordinary events to evaluate structural performance under fire exposure. For a structural system that has been conventionally designed, the following additional load combination is used to analyze its ability to endure uncontrolled fire exposure:

$$1.2D + A_k + 0.5L + 0.2S$$

$A_k$ : load effect from fire.

$D$ : load effect from dead load.

$L$ : load effect from live load.

$S$ : load effect from snow load.

The force in structural components due to fire effects has a load factor of 1.0. The live load factor of 0.5 included in the extraordinary event load combinations is intended for typical occupancies and arbitrary point-in-time live loads that will likely exist during an uncontrolled fire [47]. It is noteworthy that this live load factor represents a philosophical difference from the approach used in standard fire testing in which the assembly is loaded to its design limit for member stress, representing application of the full dead and live load. Also, this load combination excludes hurricane wind and seismic event consideration due to the negligible probability of joint occurrence with an uncontrolled fire. Granted, fire following earthquake would

have a higher probability of occurrence, and such an event has not yet been explicitly addressed within ASCE/SEI 7, or any other relevant standards.

The applicable load combination in the Eurocode includes wind loading. However, such considerations may be identified as a discretionary performance objective within the framework of Appendix E if deemed necessary by stakeholders. Since the relation between the nominal fire load (i.e., temperature at a given time) and the resulting fire load effect is usually nonlinear, the designer must apply the load factor only to the fire load effect itself, and not the nominal fire load (i.e., temperature). Conveniently, the current fire load effect factor specified in ASCE/SEI 7 is unity, so there is no procedural impact.

As it applies to uncontrolled fire exposure, the input into the load combination shown above would be the axial force, shear force, bending moment, and torque induced from restrained thermal expansion and contraction of structural members and elements. The nominal fire load (i.e., temperature at a given time) must be determined for the specific design condition (e.g., fuel load, enclosure characteristics). Hence, a trivial application of the standard fire curve (used for furnace testing) is not permitted. Currently, ASCE/SEI 7 specifies that the selection of structural design fire scenarios is within the purview of the designer. Similarly, the Eurocode does not provide specific requirements for the selection of structural design fires. However, the industry is currently developing standards that may soon assist in this selection process, and relieve some of the onus on designers in this respect. In Europe, it is relatively common for this selection to be based on some form of risk analyses, which varies from country to country and project to project. Moreover, it is typically assumed that structural design fires involve only one fire compartment of a building.

Unlike conventional structural engineering design, ASCE/SEI 7-16 Section E.6 requires the designer to consider the time and path dependence of fire effects on a structural system. While gravity loads on a structural system remain relatively constant during fire exposure (assuming most of the building contents are not burning), time-dependent temperature histories may result in time-varying member strength and thermally induced forces, depending on the temperatures reached by structural components. Hence, consideration of a specific static state may not be sufficient since preceding structural system behaviors may influence overall performance. For instance, the thermal expansion of secondary members may induce out-of-plane loads on primary members and column connections.

ASCE/SEI 7 Appendix E is referenced for structural fire engineering analyses in both the NFPA 5000 [48] and NFPA 101 [49] standards. Notably, NFPA 5000 is adopted as the governing building code in various Middle East regions. Also, NFPA 101 is adopted by some U.S. states. As a supplement to ASCE/SEI 7 Appendix E, ASCE/SEI Manual of Practice No. 138 [50] provides recommendations for analysis techniques, input parameters, and structural acceptance criteria. Also, the freely available ASCE/SEI Structural Fire Design Guide [51] explicitly demonstrates the proper application of ASCE/SEI 7 Appendix E for four anonymized steel-framed buildings. These resources in aggregate clearly demonstrate the proper interpretation and execution of ASCE/SEI 7 Appendix E provisions for structural fire engineering designs.

### ***2.2.3 Performance Expectations***

The majority of national codes default to prescriptive approaches where fire resistance is achieved by qualification against standard fire testing. Such approaches provide indeterminate performance, for they overlook key structural fire effects. Accordingly, a structural fire engineering approach is necessary to understand the level of anticipated performance.

Performance expectations relate to both occupant life safety and project-specific goals. In terms of life safety, occupants must have sufficient time and access to safely evacuate or move to an area of refuge within a given building. Accordingly, structural support of building egress routes must be maintained for a period of time necessary to ensure safe evacuation. Likewise, the building's refuge areas must remain safe for an indefinite period of time in order to maintain their design intent.

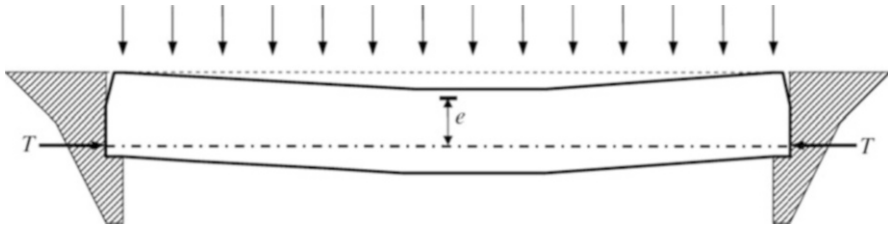
In addition to occupant life safety aspects, additional project-specific performance expectations may also be required. These are generally determined by the authority having jurisdiction and/or various stakeholders of the project. These may include concepts such as resilience: recovery of function, property protection, business continuity, environmental protection, adequate structural support of fire resistance-rated assemblies to limit fire and smoke spread, and structural support for first responders. In any case, project-specific (discretionary) performance objectives should be memorialized within a design brief (see Sect. 3.3.2).

### ***2.2.4 Restraint and Continuity***

In Sect. 2.1.4, the discussion on “restrained vs. unrestrained” focused on how structural elements are supported in standard fire testing. When the performance of a structural system is being assessed under the strength domain it is important to account for how restraint and continuity may contribute to the resistance of the structure. An explanation of how the building works in general is helpful to account for these effects.

Buildings are generally an assemblage of horizontal and vertical structural elements that are tied together. Although initial sizing of structural members may be performed considering them in isolation, the entire building works as a system, and taking advantage of this “continuity” allows redundancies to be introduced into the overall structure. The redundancy aids load sharing between members such that failure of an element may not result in failure of the entire building. In the event of a fire, structural continuity usually aids the overall performance of the building, but there are scenarios where restraint conditions may contribute to the collapse of the building.

The behavior of frames or structural systems is more complex than the behavior of individual elements, due to interactions between elements and fire-induced effects and coupling of the interactions and effects on the heated members as well as areas of



**Fig. 2.16** Effect of axial restraint force [52]

the building which may not be subject to direct heating. It is therefore important to examine the contributions of restraint and continuity in isolation. When structural elements are heated they expand in all directions. For isolated members, this expansion may occur freely and without inducing additional stresses on the particular member if the expansion is uniform across the depth of the cross section and there is no resistance to the free expansion of the member. On the other hand, for a structural member that is part of a system, the expansion of the member against cooler elements in its vicinity may increase its capacity or cause its failure.

Figure 2.16 shows a simply supported concrete beam carrying a uniformly distributed load between rigid supports. The supports allow rotation of the beam but prevent elongation. When the beam is exposed to fire at the bottom, there is differential thermal expansion through the depth of the beam, causing the bottom fibers to expand more rapidly than the top fibers. However, the presence of the rigid supports aids the development of axial thrust forces ( $T$ ) at the base of the beam, creating negative moments ( $Te$ ) that tend to resist the downward deflection induced by the differential heating through the beam's depth and the effect of the uniformly distributed load. In the figure "e" is the distance between the centroid of the beam's compressive stress region and the location of the resultant axial thrust. It is obvious from the figure that  $Te$  reduces vertical deflections as long as the thrust is below this centroid, and increases vertical displacements otherwise, as shown in Fig. 2.17(a).

The locations of several axial thrust forces are shown in Fig. 2.17. An axial thrust near the top of the beam in Fig. 2.17a would lead to failure of the beam in fire while Fig. 2.17b, c show systems that would increase the capacity of the beams. For built-in construction, where the line of action is not immediately known, as in Fig. 2.17d, the position of the axial thrust may vary. Mostly it is at the bottom, where most of the heating and thermal expansion occur.

The examples described above relate to reinforced concrete beams. In steel and composite construction, heated restrained beams develop compressive forces in their bottom flange as they expand against colder adjacent structure in the initial stages of a fire. This continues until a point at which local buckling of the bottom flange occurs, at about 300 °C as shown in Fig. 2.18. The loss of strength of the steel beam, coupled with the loss of axial thrust after buckling, induces large deflections, which eventually put the restrained beam into a catenary at temperatures above 800 °C. In Fig. 2.18, the axial forces show shortening of the beam in the cooling stage. This has

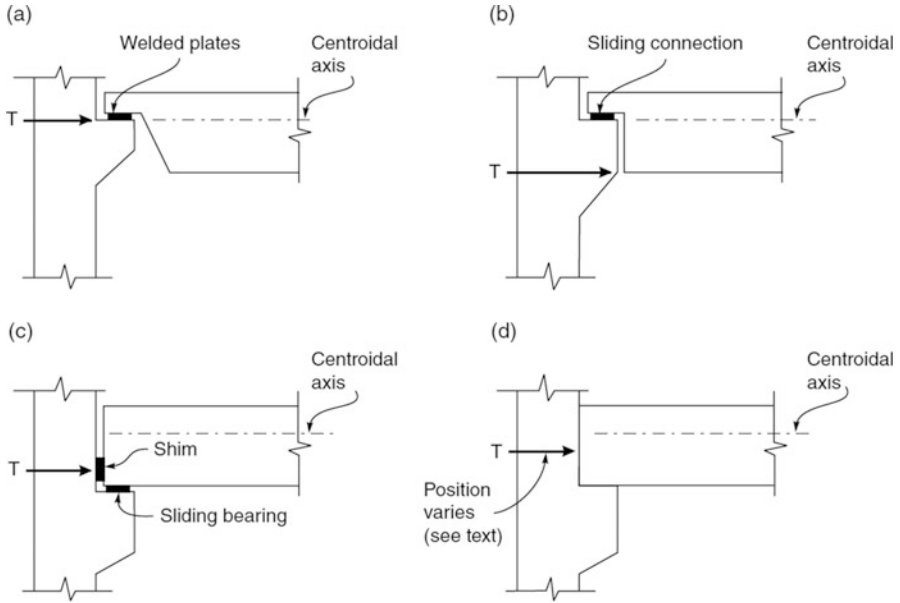


Fig. 2.17 Location of axial thrust for several support conditions [53]

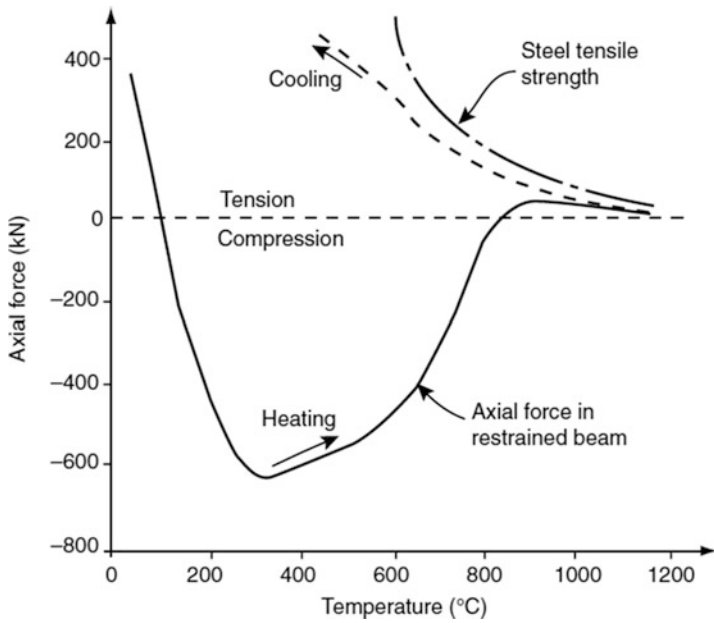


Fig. 2.18 Axial force in a restrained composite beam [54]

been known to cause failure of connections, as evidenced by the Cardington tests [55].

Restrained columns have many interactions with surrounding structure. Under fire conditions they are subject to significant changes in loading, which may affect their performance. Columns in buildings experience axial and rotational restraints. When a column is heated, it expands in all directions. When the elongation of the column is restrained, additional axial load is introduced into the column. A further increase in heating results in an increase of the induced load and reduces the column stiffness, which aids buckling. The buckling length of column is dictated by the amount of fixity at the ends of the column. As the relative fixity of the ends of the column changes throughout fire exposure, it is difficult to specify a unique design value for buckling length factor under fire conditions. Column-bending moments also change during the fire. This occurs as a result of the changing column-bending stiffness in relation to the surrounding structure. Bending moment is also affected by lateral loads induced in the column as a result of restrained thermal expansion of connected beams and p-delta effects due to the large eccentricity that occurs due to changes in initial straightness as the expansion of a connected beam is restrained.

Similar to restrained beams, restrained slabs expand against their supports. Rotational restraints along the slab boundary provide hogging (or negative) moments which help to limit vertical deflections. Axial thrusts that are at the base of the support due to the restrained thermal expansion tend to create a mechanism known as compressive membrane action. The mechanism, also known as arch action, increases the capacity of the restrained slab up to depths of the order of half the thickness of the slab. If the slab continues to experience restrained thermal expansion, then thermal buckling occurs which results in large deflections. The large deflections are only arrested if the slab can go into tensile membrane action—a mechanism that occurs in two-way bending slabs at deflections greater than the thickness of the slab. Compressive and tensile membrane action is covered in more detail in Chapters 6 and 7.

As discussed above, structural continuity has several advantages. However, in fires there are scenarios where this could contribute negatively to structural performance. The discussions above show that the negative moments at the ends of beams in particular allow them to carry more load. For a beam with doubly symmetric cross section (e.g., I-beam), positive and negative cross-section capacities are the same. In fire, as loads are maintained and structural capacities degrade, the supports yield first. The excess moments that were being carried at the supports before the loss of capacity are now redistributed to the beam midspan, which may eventually yield as the capacity of the beam reduces.

For situations where there are unequal capacities at the beam midspan and support or where the fire causes nonuniform heating along the beam, it is possible for a significant reduction in midspan beam capacity as a result of localized heating. Under this condition moments are redistributed to the supports. The behavior of the original fixed beam now becomes one representative of two cantilever beams.



### 2.2.5 Calculation Methods

Generally, the design of structures for fire effects requires the generation of fire and assessments of the thermal and mechanical responses of structures. Depending on the particular design scenario, the complexity involved in the selection of the fire and subsequent thermomechanical analysis varies. Historical models for fire exposure and corresponding structural response are presented by Purkiss and Li [56] and Buchanan and Abu [52]. Over time, there has been a shift towards large-scale, nonstandard fire testing. Generally, it is advised that when researchers carry out these tests, they should follow the accepted level of “consistent crudeness” [8]. Essentially, this principle recommends that researchers apply a similar level of crudeness to both the structural analysis in fire and the thermal insult to the structure.

Figure 2.19 shows how consistent crudeness may be applied to structural fire testing. This figure compares the complexity of the fire with the complexity of the structural analysis. The location of the intersect of the two levels of complexity indicates the credibility of the test. The intersect of the standard fire curve with a single structural element is representative of the standard fire test, used to determine

Structural Model \ Fire Model		Materials & Partial Elements	Single Elements	Sub-Frame Assemblies	Transiently Simulated Restrained Assemblies	Full-Scale Structures				
Elevated Temperature Exposures (Transient or Steady State)		Generate design/model input data	O/R	M/C	M/C	M/C				
Standard Fires		Generate design input data		O/R	M/C	M/C				
Equivalent Fire Severity to a Standard Fire		Validation of fire severity concept					M/C			
Parametrically Defined Fire Models		Generate design input data (highly dependant time-temperature phenomenon)					O/R	O/R	O/R	O/R
Numerical Fire Models		Generate design input data/ Research (highly dependant time-temperature phenomenon)					O/R	O/R	O/R	O/R
Real Fires		Research (highly dependant time-temperature phenomenon)					M/C	M/C	O/R	

**Fig. 2.19** Fire models and structural response models, where M/C indicates marginal credibility and O/R indicates occasional research [8]

structural fire resistance ratings. Comparatively, the intersect of a real fire exposure with a full-scale structure is representative of a real building fire. The diagonal connection of these two extremes represents the desired level of consistent crudeness.

Generally, the assessment of structural response to standard fire exposure is performed for single elements and subassemblies. Exposure to time-equivalent fires may be used for single elements, subassemblies, and full-frame structures while the complexity involved in modeling the effects of real fires may be applied to more complex, full-scale structural response models. Due to the growth of research in the structural fire engineering field on material behavior, the use of complex analysis is becoming more common in design. Thus a more structured design approach may be employed.

A general structural fire design typically proceeds in the following steps: First, the objectives for the design are set. Next, the required structural performance during the fire exposure must be defined, and the acceptance criteria will be determined. From this, an appropriate design fire can be selected. Member temperatures should be estimated, and the structural response of the test should be assessed. From this process, structural system characteristics can then be altered as needed, and the above process can be repeated until the design acceptance criteria are met.

This process, and in particular the steps of selecting a structural design fire, estimating the member temperatures, assessing structural response, and reassessing structural system characteristics, may require the use of a calculation method. Calculation methods may be “simple,” “advanced,” or a mixture of simple and advanced calculations. The choice of each depends on the complexity of the design problem. For example, the response of a simply supported steel beam in a small compartment may be determined by the use of the standard fire as the fire model, followed by an assessment of its thermal response using the simple lumped mass approach and then a verification of its strength loss in comparison with the applied loading.

On the other hand, the assessment of the failure of a structural connection in a composite structure under large deflections in fire conditions may require sophisticated use of an advanced fire model, an advanced thermal model, and an advanced structural model. These two examples demonstrate two extremes in the choice between simple and advanced calculation methods. However, an advanced structural analysis can also employ a simple fire model or a simple thermal model for the problem being considered, as appropriate.

After design requirements have been established, acceptance criteria are agreed upon by the stakeholders. The choice of acceptance criteria depends on the design requirements, level of conservatism to be achieved, structural design fires, familiarity of the designer with advanced calculation methods, and time allotted for analyses in the design process. In broad terms there are generally three categories of acceptance criteria: tabulated data, simple calculation methods, and advanced calculation methods.

The use of tabulated data (for most cases) does not require a consideration of the effects of applied loads at elevated temperature. Simple calculation models, on the

other hand, account for loads at the fire limit state. However, their use is limited to “simple” structural elements, as they do not account for structural continuity, load sharing, and effects of restrained thermal expansion. Advanced calculations are able to account for interactions between different structural elements, and thus provide more realistic structural behavior under fire conditions.

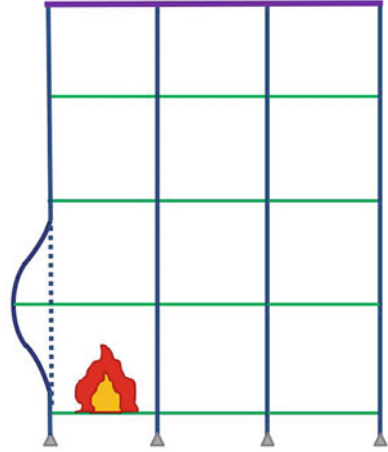
It is important to note that regardless of the selection of a calculation method the requirements of the design for fire conditions have to be satisfied. Basically, the effects of all actions at elevated temperatures must be less than the resistance that can be generated by the structure. This can simply be expressed by the inequality [57]:

$$\alpha E \leq \Phi R$$

In the equation above  $E$  is the cumulative effect of all actions on the structure. This includes permanent loads, variable loads, and effects of thermally induced actions.  $R$  is the resistance of the member.  $\alpha$  represents the resultant partial safety factor for loading at the fire limit state, which generally takes a value less than 1.0 as a result of the low likelihood of having significantly large fires and the full characteristic loads occurring at the same time, while  $\Phi$  is a partial safety factor for the given material, which typically takes a value of 1.0 for conservatism in the estimation of member response.

Single-member analyses are used to assess the response of structural elements isolated from the rest of the structure by the use of idealized boundary conditions (simple supports, pinned conditions, or fixed conditions). The analyses are usually performed by hand or by employing spreadsheets with very simple equations based on room-temperature structural analyses. Temperature distributions through the element of concern may be obtained by using tabulated data from standard fire experiments of similar structures or by simple heat transfer analyses such as the lumped temperature approaches, as outlined in Eurocode 3 for steel structures [58]. For exposure to other materials an advanced thermal analysis may be required. This typically involves the definition of the change of thermophysical properties (thermal conductivity, density, and specific heat capacity) with temperature and the use of finite differences or finite elements to solve for the member temperatures over the duration of the fire. It is important to note that single-member analyses may not sufficiently capture the effects of the thermal exposure (thermal expansion in all three directions and the effects of restrained thermal expansion or curvature), due to the simplicity of the problem setup. As such, it is recommended that these analyses are used for scenarios where the additional thermal induced effects are negligible.

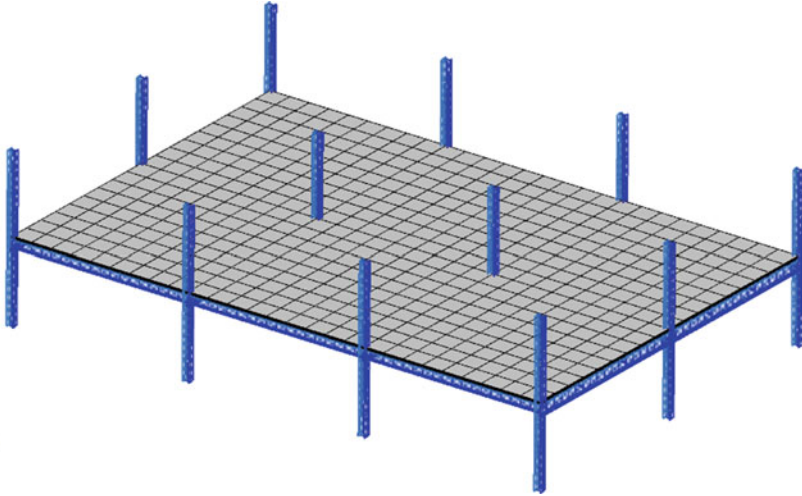
Sub-frame analyses are typically two-dimensional in nature. An example of a sub-frame is shown in Fig. 2.20, which shows a two-dimensional frame of a building exposed to in-plane actions (p-delta effects). Due to the interactions between the elements a more realistic structural behavior may be captured with this structure than can be obtained from the single-element analyses. Element temperature distributions throughout the structure may be obtained similarly to what is described for single-member analysis. The two-dimensional setup allows the frame to account for the effects of restrained thermal expansion in the plane of the frame. Additional

**Fig. 2.20** Sub-frame model

considerations include nonlinear material behavior and geometric nonlinear behavior. The frame can also be used to investigate the stability of the structure as well as explore alternative load paths as parts of the frame lose strength and stiffness. However, it is unable to capture the effects of thermal expansion or restraints in the direction perpendicular to the plane of the frame. It should be noted that fires burn in a three-dimensional space, and the transfer of heat throughout the structure is also three-dimensional. This implies that the use of 2D sub-frames as shown in Fig. 2.20 does not adequately account for loads or the behavior of the structure in the third dimension [59]. Thus, particular care should be taken when sub-frames are being used for analyses.

As observed from the previous section, sub-frame models are deficient in structural fire engineering analyses as thermal and mechanical effects in the third (perpendicular) direction cannot be accounted for. Thus full-frame analyses offer the solution to the problem. A typical full-frame model is shown in Fig. 2.21.

The effect of a fire that occurs underneath any of the structural bays shown in Fig. 2.21 can suitably be interpreted in terms of the thermal exposure of all elements in its vicinity (including slabs). For typical fully developed fire exposures it is assumed that the structural elements only experience temperature variations in their cross section and not along their axes. However, these may be modified when localized fires or fires that “travel” across a floor plate are encountered. The inclusion of the third dimension also allows slab effects to be considered in the analysis. This produces more representative behavior of a structure under fire conditions. The structural analyses are performed by the use of finite elements, as a number of thermophysical and mechanical properties need to be incorporated into the model.



**Fig. 2.21** Full-frame model

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