12 FIRE RESISTANT DESIGN

12.1 DESIGN PRINCIPLES

This section aims to provide guidance on fire resistant design in steel and composite structures which deals primarily with minimising the risk of structural collapse and restricting the spread of fire through the structure.

The fire resistant design method is applicable to steel and composite structures with the following materials:

Structural steel:	Hot rolled steel sections with design strengths equal to or less than 460 N/mm^2.
	Cold formed steel sections with design strengths equal to or less than 550 N/mm^2.
Concrete:	Normal weight concrete with cube strengths equal to or less than 60 N/mm^2.
Reinforcement:	Cold worked reinforcing bars with design strengths equal to or less than 500 N/mm^2.

For steel materials other than those listed above, refer to specialist design recommendations. Alternatively, passive fire protection method should be adopted.

12.1.1 Basic requirements of fire resistance for a structure

a) Mechanical resistance

All structural members in a structure shall be designed and constructed in such a way that integrity failure of the structure does not occur and their load bearing capacities are maintained at the end of the specified fire resistance period.

b) Compartmentation

All those members of a structure forming the boundaries of a fire compartment, including joints, shall be designed and constructed in such a way that they maintain their separating function during the specified fire resistance period, i.e. no integrity nor insulation failure shall occur.

Whenever a fire test is carried out, the fabrication and interface details should follow the details to be used on site as fire testing is essentially a system integrity test.

12.1.2 Fire exposure

The fire exposure to a structure may be based on either of the following two types:

a) Standard fire

The standard fire is a controlled fire exposure described by a standard time - temperature curve as follows:

$$T = 345\log_{10}(8t+1)+20 \qquad (12.1)$$

where

T = design fire temperature (^{o}C)

t = elapsed time (minutes)

The required fire resistance periods (FRP) shall be established according to the requirements as stipulated in the current Code of Practice for Fire Resisting Construction.

b) Natural fire
A natural fire refers to a fire exposure that builds up and decays in accordance with the mass and energy balance within a compartment. The quantification of a natural fire shall be performed according to established methods, taking into consideration of uncertainties and risks.

12.1.3 Fire limit states

The structural effects of a fire in a building, or part of a building, shall be considered as a fire limit state, and a fire limit state shall be treated as an accidental limit state. At the fire limit state, a structure shall maintain the integrity of fire resistant compartments for the specified fire resistance period under the entire period of a standard or a natural fire. This shall be achieved by ensuring that

- all structural members in the structure can maintain their load carrying capacities, or
- the loss of load carrying capacity of any member does not cause integrity failure of the fire resistant compartmentation.

Whenever the load carrying capacity of a structural member is demonstrated to be sufficient through the use of advanced methods involving large deformations, it is necessary to ensure that there is no integrity failure at junctions of different structural members caused by these large deformations. Moreover, requirements for insulation and integrity of compartment walls and floors, including any incorporated members, shall also be satisfied.

The fire performance of structural members or a structure shall be determined either through standard fire tests on loaded members or through fire resistant design based on established design methods.

Whenever fire protection materials are required to achieve the specified fire resistance period, the thicknesses of the fire protection materials should be derived from standard fire tests at accredited laboratories whilst the recommendations should be prepared by a suitably qualified person. Alternatively, current assessment methods include (a) standard fire tests, (b) limiting temperature methods, (c) performance-based design methods, and (d) simplified calculation methods.

Connection plates, stiffeners and similar elements should be ordinarily treated with the same fire protection thickness as the primary steel member to which they are attached.

Bracing members required to ensure stability to a structure at the fire limit state shall remain functional during the specified fire exposure, unless alternative load paths can be identified in the structure. Whenever practicable, bracing members should be built into other fire resisting components of the building, such that the bracing members need no additional protection.

12.1.4 Physical and mechanical properties at elevated temperatures

Table 12.1 summarizes all the physical properties of steel and concrete at elevated temperatures for simplified thermal calculations.

Data on the strength reduction factors of hot rolled and cold formed steel, normal weight concrete, and cold worked reinforcing bars at elevated temperatures shall be obtained from Table 12.2. Appropriate level of strains shall be adopted for various types of construction.

For bolts and welds, the strength reduction factors shall be taken from Table 12.3.

Strength reduction factors are used for checking load carrying capacities of structures under different loading and temperature conditions, with or without uses of fire protection materials for various degrees of insulation against outside temperature.

Table 12.1 - Typical physical properties of steel and concrete at elevated temperatures (for simplified thermal calculations)

Property	Steel	Normal Weight Concrete
Density (kg/m^3)	7850	2300
Specific heat (J/kg °C)	600	1000
Thermal conductivity (W/m °C)	45	1.6
Coefficient of expansion (x 10^{-6})	14	18
Poisson's ratio	0.3	0.2

Table 12.2a - Strength reduction factors for hot rolled steels at elevated temperatures

Temperature (°C)	Strength reduction factors		
	0.5 % strain	**1.5 % strain**	**2.0 % strain**
20 °C	1.00	1.00	1.00
100 °C	1.00	1.00	1.00
200 °C	0.92	1.00	1.00
300 °C	0.84	0.99	1.00
400 °C	0.76	0.98	1.00
500 °C	0.61	0.76	0.78
600 °C	0.35	0.46	0.47
700 °C	0.17	0.23	0.23
800 °C	0.09	0.11	0.11
900 °C	0.06	0.06	0.06
1000 °C	0.04	0.04	0.04
1100 °C	0.02	0.02	0.02
1200 °C	0.00	0.00	0.00

Table 12.2b - Strength reduction factors for cold-formed steels at elevated temperatures

Temperature (°C)	Strength reduction factors
20 °C	1.00
100 °C	1.00
200 °C	0.89
300 °C	0.78
400 °C	0.65
500 °C	0.53
600 °C	0.30
700 °C	0.13
800 °C	0.07
900 °C	0.05
1000 °C	0.03
1100 °C	0.02
1200 °C	0.00

Note: In the absence of any other information, Table 12.2b may be adopted for cold-formed structural hollow sections.

Table 12.2c - Strength reduction factors for normal weight concrete at elevated temperatures

Temperature (°C)	Strength reduction factors
20 °C	1.00
100 °C	1.00
200 °C	0.95
300 °C	0.85
400 °C	0.75
500 °C	0.60
600 °C	0.45
700 °C	0.30
800 °C	0.15
900 °C	0.08
1000 °C	0.04
1100 °C	0.01
1200 °C	0.00

Table 12.2d - Strength reduction factors for cold worked reinforcing bars at elevated temperatures

Temperature (°C)	Strength reduction factors
20 °C	1.00
100 °C	1.00
200 °C	1.00
300 °C	1.00
400 °C	0.94
500 °C	0.67
600 °C	0.40
700 °C	0.12
800 °C	0.11
900 °C	0.08
1000 °C	0.05
1100 °C	0.03
1200 °C	0.00

Table 12.2e - Strength reduction factors for hot rolled reinforcing bars at elevated temperatures

Temperature (°C)	Strength reduction factors
20 °C	1.00
100 °C	1.00
200 °C	1.00
300 °C	1.00
400 °C	1.00
500 °C	0.78
600 °C	0.47
700 °C	0.23
800 °C	0.11
900 °C	0.06
1000 °C	0.04
1100 °C	0.02
1200 °C	0.00

Table 12.3 - Strength reduction factors for bolts and welds at elevated temperatures

Temperature (°C)	Strength reduction factors for bolts	Strength reduction factors for welds
20 °C	1.00	1.00
100 °C	0.97	1.00
150 °C	0.95	1.00
200 °C	0.94	1.00
300 °C	0.90	1.00
400 °C	0.78	0.88
500 °C	0.55	0.63
600 °C	0.22	0.38
700 °C	0.10	0.13
800 °C	0.07	0.07
900 °C	0.03	0.02
1000 °C	0.00	0.00

12.1.5 Material factors and load factors

In checking the strength and stability of a structure at the fire limit state, the following material and load factors shall be adopted:

Table 12.4 - Material factors for fire limit state

Material	γ_m
Steel	1.00
Concrete	1.10
Reinforcement	1.00

Table 12.5 - Load factors for fire limit state

Loads	γ_f
Dead load	1.00
Imposed loads:	
a) permanent:	
1) those specifically allowed for in design, e.g. plant, machinery and fixed partitions	1.00
2) in storage buildings or areas used for storage in other buildings (including libraries and designated filing areas)	1.00
b) non-permanent:	
1) in escape stairs and lobbies	1.00
2) all other areas (imposed snow loads on roofs may be ignored)	*0.80
Wind loads	0.33

Note: * The value may be reduced to 0.50 when suitable justification is available.

Whenever appropriate, the effect of wind load should be considered in checking primary members of a structure. However, there is no need to check for secondary members of the structure under the effect of wind load except in very special circumstances.

12.2 FIRE RESISTANCE DERIVED FROM STANDARD FIRE TESTS

In general, it is considered that all the construction elements specified in the Code of Practice for Fire Resisting Construction are considered as satisfying the specified fire resistance periods. For other construction elements and structural members, they should be tested in accordance with appropriate fire testing requirements to verify their fire resistance periods.

12.2.1 Fire resistances of structural members

The fire resistances of structural members shall be established by:

- a test report indicating that the construction elements or the structural members are capable of resisting the action of fire for the specified period. The test should be carried out and the test report prepared and endorsed by a HOKLAS accredited laboratory or other accredited laboratory which has mutual recognition agreements / arrangements with the HOKLAS or the BA; or
- when the complexity and scale of fire testing is not practical, an assessment report against established standard fire testing that the construction elements and the structural members are capable of resisting the action of fire for the specified period. The assessment should be carried out and the assessment report prepared and endorsed by:
  - i) a HOKLAS accredited laboratory or other accredited laboratory which has mutual recognition agreements / arrangements with the HOKLAS or the BA; or
  - ii) an establishment or a professional having the appropriate qualifications and experience in fire resisting construction recognized by the BA.

The loads applied in these fire tests shall correspond to the design factored loads at the fire limit state. Where the design factored loads for the fire limit state differ from those applied in the tests, the test results should be adjusted according to established design recommendations. These tests shall be carried out at accredited laboratories whilst the recommendations derived from them shall be prepared by a suitably qualified person. For details, refer to the Code of Practice for Fire Resisting Construction 1996.

12.2.2 Failure criteria for standard fire tests

The fire resistance of a structural member shall be determined with respect to load-bearing capacity, integrity and insulation as follows:

12.2.2.1 Load-bearing capacity

The test specimen shall be considered to have failed in load-bearing capacity if it is no longer able to support the test load. This shall be taken as either of the following, whichever is exceeded first:

a) For flexural members:

(i) Deflection limit = $\dfrac{L^2}{400\,d}$ (in mm); and (12.2a)

(ii) Rate of deflection = $\dfrac{L^2}{9000\,d}$ (in mm/min) when a deflection of L/30 has been exceeded. (12.2b)

where L is the clear span of specimen (in mm); and

d is the section depth of the structural member (in mm).

b) For axially loaded compression members:

(i) Axial shortening limit = $\frac{h}{100}$ (in mm); and (12.3a)

(ii) Rate of shortening = $\frac{3h}{1000}$ (in mm/min) (12.3b)

where h is the initial height (in mm).

12.2.2.2 Integrity

The test specimen shall be considered to have failed in integrity when any one of the following occurs:

- collapse,
- sustained flaming on the unexposed face lasting more than 10 seconds,
- ignition of a cotton pad, or
- opening of an excessive gap.

12.2.2.3 Insulation

The test specimen shall be considered to have failed in insulation when one of the following occurs:

a) if the mean unexposed face temperature increases by more than 140°C above its initial value; or

b) if the temperature recorded at any position on the unexposed face is in excess of 180°C above the initial mean unexposed face temperature.

12.3 FIRE RESISTANCE DERIVED FROM LIMITING TEMPERATURE METHOD

a) This method is applicable to the determination of the fire resistance of steel members such as tension members, columns and beams.

No protection is needed when the design temperature does not exceed the limiting temperature under a specific load ratio where:

- the load ratio is defined as the load applied at the fire limit state divided by the load causing the members to fail under normal conditions, and
- the limiting temperature of a structural member is defined at the temperature at which it will fail.

b) Limiting temperatures of critical elements in various structural members under different load ratios shall be obtained from established recommendations.

For information, the section factor, H_p / A, is defined as the ratio of the surface perimeter exposed to radiation and convection, H_p, to the cross-sectional area, A.

12.4 FIRE RESISTANCE DERIVED FROM PERFORMANCE-BASED DESIGN METHODS

12.4.1 Basis of analysis

a) Based on fundamental physical behaviour of materials and behaviour of structural elements and systems, performance-based design methods provide realistic assessment of structures under fire, i.e. a reliable approximation of the expected structural behaviour of relevant individual members, subassemblies or entire structures at specific fire resistance periods. Generally, it consists of two separate calculation models for the determination of:

   (i) Thermal response

   The development and distribution of the temperature within structural elements.

   (ii) Mechanical response

   The mechanical behaviour of the structure or of any part of it.

Caution should be taken for any potential failure modes which have not been covered by the performance-based design methods, for example, local buckling, insufficient rotation capacity, spalling and failure in shear. Appropriate means such as steel constructional details should be provided to ensure that such failure modes do not control in reality.

12.4.2 Thermal response

a) The thermal response model shall be based on established principles and assumptions of the theory of heat transfer, and the temperatures shall be evaluated according to established calculation procedures.

b) The thermal response model shall consider

   (i) the relevant fire exposure in the structure;

   (ii) the variation of the thermal properties of structural elements and relevant fire protective materials within the temperature ranges considered; and

   (iii) appropriate boundary conditions.

c) The effects of non-uniform thermal exposure and of heat transfer to adjacent building components shall also be incorporated wherever appropriate.

d) The effects of active protective measures, such as detection or sprinkler system, may be included wherever appropriate.

e) For simplicity, the influences of moisture content and migration within concrete and fire protection materials may be conservatively neglected.

12.4.3 Mechanical response

a) The mechanical response model shall be based on established principles and assumptions of the theory of structural mechanics, taking into account the effects of temperature. It should incorporate:

   (i) the combined effects of mechanical actions, geometrical non-linear effects, geometrical imperfections and thermal actions;

   (ii) the stress-strain curves at elevated temperatures;

   (iii) the effects of non-linear material properties, including the effects of unloading on the structural stiffness; and

   (iv) the effects of thermally induced strains and stresses, both due to temperature rises and differentials.

b) The deformations at fire limit state should not be excessive in order to maintain integrity and compatibility between all parts of the structure. However, the effects of high temperature creep may not be considered explicitly.

12.4.4 Validation of performance-based design methods

a) The calculation results shall be verified against relevant test results in terms of deformations, temperatures and fire resistance periods.

b) A sensitivity analysis on the calculation results shall be carried out against all critical parameters required in the verification such as the buckling length, the size of the elements, the load level and etc., in order to ensure that the model complies with sound engineering principles.

12.4.5 Simplified calculation methods

a) For structural adequacy, the moment capacity of a beam member such as a steel beam, a composite beam, or a composite slab with profiled steel decking at elevated temperatures should be greater than the applied moment in fire.

b) The moment capacity of the beam member should be evaluated according to rectangular stress blocks comprising the reduced strength of various elements of the member at elevated temperatures.

c) Material properties, reduction factors for various materials at elevated temperatures, and load factors in fire limit state given in Tables 12.1, 12.2, 12.3, 12.4 and 12.5 should be adopted.

d) For other types of structures, such as external structures exposed to external fire exposure, water-filled tubes and portal frames, established design methods should be followed.

e) If fire protection is specified in any of the connecting members, the same protection system should be provided to the connection regions.

12.5 PERFORMANCE REQUIREMENTS

Structural members and systems should be designed to resist external loads under fire exposure with partial load factors for extreme events obtained from Table 4.3. Deformation should be limited to the following limits: -

i) for flexural members, deflection limit = $\dfrac{L^2}{400\,d}$

ii) for axially loaded members, deflection limit = $\dfrac{h}{100}$

where L is the clear span of specimen (in mm);

d is the section depth of the structural member (in mm); and

h is the initial height (in mm).

Advanced analysis allowing for thermal and mechanical properties of structural elements under fire exposure should be carried out to demonstrate the stability and integrity of structural element under fire exposure. The advanced analysis should follow the requirements as stipulated in section 6.9 with allowance for geometric imperfections and material non-linear effects. Degradation of strength and stiffness of the structural systems at elevated temperature, with or without fire protection, should be allowed for in the advanced analysis.