Fire Engineering Application to Multi-Story Steel Structures

by G. Charles Clifton and Martin J. Feeney

Guidelines and suggestions for keeping structures and their occupants safe during a fire event.

What is the appropriate design procedure to check a building's ability to withstand a fully developed fire attack? It is important to understand a multi-story steel building's response to fire and elevated temperature. When an individual steel beam reaches its limiting temperature, it still has enough strength to support the design long-term load indefinitely and without excessive deflection. The majority of beams over a given area will not suffer local buckling of the cross-section, with the possible exception of smaller members that are highly restrained by larger members.

Recent open-parking-structure fire tests were conducted on a representative section, where the floor system above the fire was loaded to approximately the fire-emergency design load. The supporting steel beams had a limiting steel temperature of about 650 °C. In this case, maximum steel temperatures were only 350 °C, so no permanent effect on the steel structure was expected or resulted.

For beams that just reach their limiting temperature, some thermal-induced downward deflection from the fire is likely to remain. Beams in a structural system gain rotational restraint from the surrounding members, regardless of the end connection used. If the restraint from simple or semi-rigid connections is ignored when calculating the limiting temperature, then the permanent deflection, post-fire, associated with reaching the limiting temperature is minimal. The effects of this beam sag on any components supported by the beam will be negligible compared to the fire's effects. Once cooled, the structure will have the same strength as before the fire. Cracking in the top of a concrete floor slab above the fire floor is usually of aesthetic concern and readily repairable.

In a fire test for the 140 William Street Office Building in Melbourne, Australia, fire load was high (e_f = 1260 MJ/m^2 floor area) but ventilation conditions were good (OF = 0.22 m$^{0.5}$). The test structure was a mock-up of the existing high-rise building, and the castellated floor-support beams over the enclosure supported a concrete floor slab carrying a superimposed live load of 3.74 kPa. This loading and structural layout gave castellated secondary floor beams a limiting temperature near 520 °C.

The test result was a short duration period of fully developed fire, with peak gas temperatures below the suspended ceiling of more than 1200 °C for around 5 min. and more than 600 °C for 15 to 18 min. The suspended ceiling remained largely in place and functioned as a radiation barrier, keeping maximum temperatures in the castellated beams to 535 °C.

The peak mid-span deflection in the beams was 120 mm, with most recovered when the fire and building cooled to ambient temperature. If the steel temperature is raised above the beam-limiting temperature associated with simple end-support conditions, then the beam's stiffness and strength reduce further, and load is redistributed to cooler parts of the structure that are stiffer and stronger.

In this case, larger beam-sagging deflections and local buckling of the beam cross-section could occur, but in most steel buildings, the loads applied to the structure will continue to be supported because of structural continuity and redundancy. The steel members subject to

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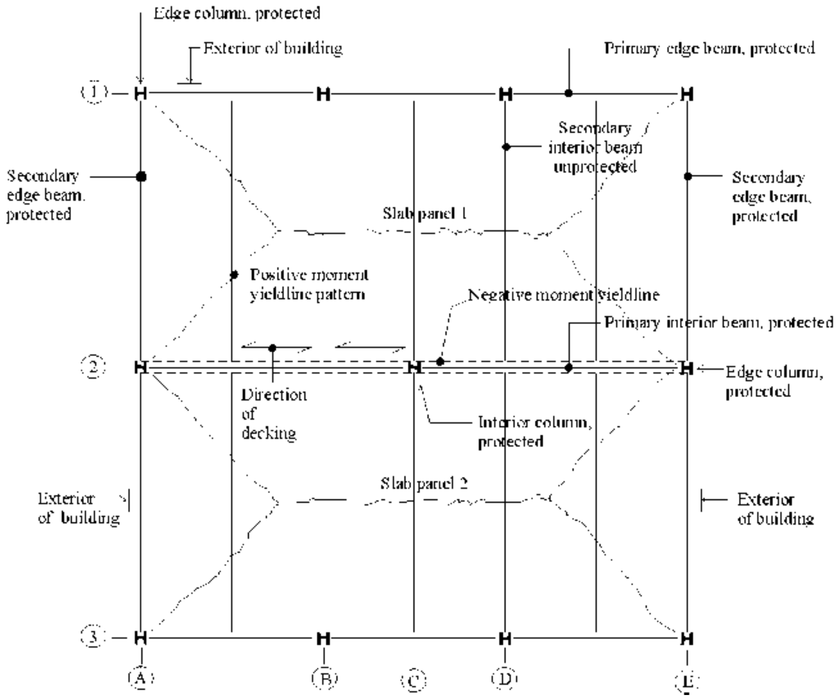

Reflected floor plan for application of slab-panel fire-engineering design procedure to a concrete slab on profiled steel deck, supported on primary and secondary beams.

the highest temperatures might sustain local permanent damage, but experience shows that they are repaired or replaced readily, especially if inelastic demand is restricted to the beams and suppressed in the columns.

The load sharing from continuity and redundancy allows elements in the structure to be heated beyond their limiting temperature without affecting the structure's overall strength and stability. This also accounts for the better performance of multi-story steel structures in actual fires than what would be expected from the results of standard fire tests on individual members.

This inelastic reserve of strength is illustrated by the Cardington fire-test series. In this test, all beams were unprotected and columns protected. The fire was severe, with $e_f = 750$ MJ/m^2 floor area, restricted ventilation and lightweight concrete used in the floor slabs. The fire had a fully developed duration (temperatures above 600 °C) of more than 60 minutes, with peak fire temperatures above 1100 °C. Peak steel temperatures reached 1100 °C. The floor system suffered deformation but retained load-carrying capacity and integrity.

Unprotected steel members directly exposed to a fully developed fire will heat rapidly. Any unprotected elements that receive direct radiation and are not directly in contact with a heat-sink material like concrete will be subject to a time-temperature history only slightly less severe than that of the fire.

So for designs incorporating steel members without applied passive fire protection, the appropriate design procedure is dependent on:

1. Does a radiation barrier shield the member from direct exposure to the fire, and, if so:
   - How long will that barrier remain in place?
   - What is the fire time-temperature history after that barrier is removed?
2. If the member is directly exposed to the fire, and mostly not connected to a heat sink, what is the expected maximum fire temperature and duration?

Building Performance Criteria to be met in Fire Engineering Design

Mandatory provisions specify the minimum performance requirements that buildings must meet in a fire:

1. There shall be a low probability of the building suffering a loss of amenity due to fire start, or suffering collapse or structural instability under fully developed fire conditions.
2. People shall be safeguarded from injury or illness when evacuating during a fire.
3. Fire service personnel shall be protected for an adequate time during rescue and firefighting operations.
4. The neighbor shall have a high degree of protection from the effects of fire, including damage caused by structural collapse as a consequence of the fire.
5. The environment shall be safeguarded in situations where hazardous substances would be released during a fire.

A fire start can be considered as violating the serviceability limit-state condition, since it results in a loss of amenity, even though the structural system of the building is not involved. Although the return period associated with a fire start in a building varies considerably with building type, for a typical medium-rise commercial office building, a figure of one fire start in 20 years is realistic for New Zealand. This is similar to that associated with the serviceability limit-state earthquake or wind force.

If conditions are favorable for fire growth, a fire will reach full development at its point of origin. This is rare, but statistical records indicate that when it occurs, fire spread beyond the room of origin is likely. A fully developed fire in a building is an ultimate limit-state event that necessitates extensive post-fire inspection and reinstatement of structural and non-structural elements, comparable to the impact of an ultimate limit-state earthquake.

For a medium-rise commercial office building, the probability of a fully developed fire occurring over the 50-year design life in an unsprinklered building is estimated at 50%. This is considerably higher than the expected probabilities of occurrence over 50 years of the design ultimate limit-state earthquake (10%), wind (13%) or peak vertical loading (5%). But the probability of a fully developed fire occurring over the 50-year life of a sprinklered multi-story office building is less than 0.5%.

In a fully developed fire in an unsprinklered multi-story building, the structural response must be as follows:

1. Local and global collapse should be minimized, and must be prevented where it could threaten escape routes

or cause fire spread to adjacent property.

2. Structural elements that are fire separations must maintain their integrity, stability (load-carrying capacity) and insulation for either a specified time or for burnout of the fire cell.
3. Permanent damage to structural elements of the structure and to fire separations should be minimized. When non-fire-rated ceilings shield beams from direct fire exposure, they can reduce the beam's potential inelastic response.

If sprinklers in a sprinklered, multi-story building do not operate during a fully developed fire, the structural response must be as follows:

1. Global collapse must be prevented. Local collapse should be minimized, and prevented where it could lead to global collapse, threaten escape routes, or cause fire spread to adjacent property.
2. Structural elements that are fire separations must maintain integrity and stability for the fire burnout condition, and maintain a lower insulation requirement than necessary for unsprinklered buildings at least as long as the evacuation time. Given that life-endangering collapse is prevented, the maximum insulation rating required is 60 minutes.
3. If insulating fire protection is not provided to steel structures, inelastic response of the beams and floor systems is expected, and these elements must be designed and detailed accordingly. Inelastic response of the columns will be minimized to control local or global sway/collapse mechanisms and to suppress structural damage remote from the fire floor.

Slab Panel Method (SPM)

Floor support beams can be left unprotected through application of the Slab Panel Method (SPM) of floor-system design. SPM is a high-temperature design procedure applied to large regions of a floor that incorporates inelastic response. It is based on unprotected steel beam elements not in contact with concrete, reaching temperatures above 850 °C.

SPM incorporates the inelastic reserve of strength in multi-story steel-framed buildings with protected columns in severe fire conditions. It is conservative to apply it to low and moderate fire-load conditions, where lower levels of inelastic demand must be resisted. The method represents a major advance in fire-engineering design of steel-framed floor systems.

Under ambient temperature conditions, the beams support the floor slab. The load path involved in resisting dead and live loads under ambient temperature conditions is:

$$\text{Slab} \rightarrow \text{secondary beams} \rightarrow \text{primary beams} \rightarrow \text{columns} \quad (1)$$

Under severe fire conditions, when the secondary interior beams are unprotected, they lose most of their strength, and the ambient-temperature load path cannot be maintained. Beams form plastic hinges and the load-carrying mechanism changes to a two-way system, involving the region of slab and unprotected beams known as a slab panel. The slab panel resists applied load by two-way action back to the supports, through a load path involving:

$$\text{Slab panel} \rightarrow \text{supporting beams} \rightarrow \text{columns} \quad (2)$$

The same concept is applicable to floor slabs supported on closely spaced joists. The slab panel develops its load-carrying capacity in the deformed state through yield-line moment action plus tensile-membrane enhancement. The supporting beams must resist the loads transferred from the slab panel and send them back to the columns.

Colin Bailey of the UK Building Research Establishment developed SPM and the design model for yielding-moment action and tensile-membrane enhancement. Bailey validated the model with the Cardington fire tests and performed a large-scale test that verified that the internal actions developed are consistent with the model assumptions under ambient temperature conditions. This involved constructing a slab panel with 9.5 m-by-6.5 m centerline-to-centerline support dimensions, formed by casting the floor system onto a profiled steel-deck base that was removed prior to loading. The slab panel was reinforced with shrinkage and temperature-control mesh. The self-weight of the slab panel equaled its calculated yield-line load-carrying capacity, using the actual mechanical properties of the mesh. Any additional load-carrying capacity was due to tensile-membrane action and a minor contribution from strain hardening of the reinforcement.

The loading pattern and application induced effective uniform applied load. The imposed load was applied in increments. At an applied load of 0.8 times the yield-line load-carrying capacity, the cracking pattern associated with tensile-membrane action started to form. This pattern was fully developed at an applied load of 0.96 times the yield-line load-carrying capacity (a total load of 1.96 times the self-weight). The slab panel did not collapse, and the loading mechanism ran out of stroke—no further load could be applied.

There was comprehensive monitoring of stresses, strains and deflections. These results and those of subsequent ambient temperature tests from the UK confirmed the basis of the design procedure under ambient temperature, in terms of the two structural mechanisms involved and their sequence of operation.

In 2002, Lim conducted a series of standard fire tests on six 4.15 m-by-3.15 m slab panels. He tested three 100 mm-thick flat slabs with different levels of reinforcement: two slabs on profiled steel deck and one slab on cold-formed steel joints. Each slab carried its own weight plus an applied loading of 3.0 kN/m^2, delivered through water-filled drums. All slabs delivered integrity and stability for three hours of ISO standard fire exposure.

Fire tests generate information that ambient-temperature tests cannot provide: the performance of a slab panel under the ultimate limit-state environment of constant vertical load, severe fire conditions of high temperatures and thermal gradient development. But verification of structural mechanisms within the slab panel can be recorded only by measuring failure under ambient temperature loading. The same cannot be done with fire tests, since strain gauges cease to give reliable readings at 30 to 40 min. into the tests, corresponding to limiting temperatures of 300 °C and well short of slab-panel failure. Therefore, analytical modeling is required to track strains and corresponding stresses through to slab-panel failure in fire conditions.

Both N. Mago of HERA and Lim modeled five of the test panels analytically, with comparative mid-span deflections presented. These results, obtained using ABAQUS, demonstrated as close an agreement as possible between the experimental test and analytical model with regard to the mid-span vertical deflection. This was used to compare other parameters, such as reinforcing bar strains and forces.

When a slab panel is loaded to destruction through ambient temperature loading, yield-line behavior is developed first, followed by tension-membrane enhancement. However, under fire conditions, thermal bowing predominates the early stages, causing tensile-membrane enhancement to occur first, followed by both mechanisms operating concurrently.

In the event of fully developed fire conditions, SPM performs as follows:

1. Slab and unprotected secondary beams could undergo appreciable permanent deformation, up to as much as short span/15. In practice, the inelastic demand is usually less, due to:
   - Lower fire load
   - Presence of shielding linings
   - Non-fire-rated enclosures reducing the fire size
   - Fire service intervention
2. Support beams and columns undergo minimum permanent deformation, compared to that within the slab panel
3. Load-carrying capacity and integrity of the floor system are preserved.
4. Insulation requirements are met for at least the F-rating times specified by the Acceptable Solution (C/AS1, 2001).
5. Local collapse and global collapse are prevented.

In practice, structural repair of a steel building designed to SPM is comparable to that of a building with all floor support beams protected.

Effective compartmentation should be maintained between floors and fire cells. The former is part of SPM's floor-system performance. But fire-separating walls between firecells on the same floor fall into two cases: 1. For walls under slab-panel support beams, the deformation will be as expected from a system with protected beams, and standard details apply; 2. For walls that run across the slab panel, special allowance for deformations within the panel must be made.

Appropriate details to allow elements to deform without loss of function include:

- material requirements for all elements
- extent of passive fire protection required to columns, slab-panel edge supports
- connections between beams and beam-to-column
- interconnection between floor slab and beams
- reinforcement around slab-panel edges

The SPM procedure incorporates the reserve of strength from a floor system under deformation in a fully developed fire attack. It is an ultimate limit-state design procedure, similar to building design for limited- to fully ductile response to earthquake. ★

*This paper has been edited for space considerations. To learn more about fire engineering, read the complete text online at **www.modernsteel.com** or in the 2004 NASCC Proceedings.*