ROBUSTNESS OF MULTI-STOREY STEEL-COMPOSITE STRUCTURES UNDER LOCALISED FIRE

Cheng Fang
BEng, MSc

March 2012

A thesis submitted in fulfilment
of the requirements for the degree of
Doctor of Philosophy of
Imperial College London

Department of Civil and Environmental Engineering
Imperial College London
London SW7 2AZ
Declaration

I confirm that this thesis is my own work and that any material from published or unpublished work from others is appropriately referenced.
ACKNOWLEDGEMENTS

The work presented in this thesis has been carried out under the joint supervision of Prof. Bassam A. Izzuddin, Prof. Ahmed Y. Elghazouli, and Prof. David A. Nethercot. I wish to express my thanks for their excellent guidance and support. Particularly, I want express my deepest gratitude to my principal supervisor, Prof. Bassam A. Izzuddin, whose patience, constructive comments, and constant encouragement have been invaluable throughout the duration of my study, and the project meeting travels with him were quite fruitful and enjoyable.

I would also like to express my gratitude to the financial support of Research Fund for Coal and Steel of the European Community within project ROBUSTFIRE: “Robustness of Car Parks against Localised Fire”, Grant No RFSR-CT-2008-00036. The related discussions and input of our collaborators from the University of Liège, University of Coimbra, Arcelor Profil Luxembourg, CSTB, Greisch Ingénierie, and CTICM are gratefully acknowledged.

Furthermore, I wish to thank my parents, He Fang, and Kang Chen, whose sacrifices over 20 years have ensured a warm family environment. Their love and support has always been an enormous source of inspiration and strength for me.

Last but not least, I want to thank my colleagues and friends who have worked with me in room 317A and in Skempton Building over the years for their unforgettable help and company.
ABSTRACT

While current assessment methods for preventing progressive collapse are mainly associated with blast and impact loading, no systematic framework is currently available for the practical and rational assessment of robustness of multi-storey building structures under localised fire. In this thesis, a robustness assessment framework with various alternative levels of sophistication is presented with the aim of bridging the gap between the topics of structural fire resistance and progressive collapse. The robustness assessment framework developed in this thesis is comprised of four basic components, namely, detailed Temperature-Dependent Approach, simplified Temperature-Dependent Approach, Temperature-Independent Approach, and practical design recommendations. These assessment approaches can satisfy various design requirements in different design stages. To illustrate their application, localised fire induced by burning vehicles in a typical multi-storey steel-composite car park is considered as a main reference scenario. A Robustness Limit State (RLS) is proposed, which is based on the fact that large inelastic deformations of building structures subject to extreme loading are typically concentrated in the joint regions, thus failure of joints in certain locations may lead to floor collapse, and subsequently trigger progressive collapse. Therefore, no floor collapse is allowed in the current RLS. Since joint resistance and ductility play an essential role in mitigating progressive collapse, a component-based joint modelling technique and multiple joint failure criteria for commonly used semi-rigid joints are proposed and thoroughly discussed. The proposed joint modelling strategy is shown to be capable of capturing realistic nonlinear behaviour of joints under both ambient and elevated temperatures. Employing the proposed joint failure criteria, the application of the robustness assessment framework is illustrated for the reference structure, and important conclusions are drawn relating to the accuracy and reliability of each assessment approach. Furthermore, multi-level structural modelling strategies, the significance of structural dynamic effects during fire, factors influencing structural robustness, and future research topics for the avoidance of fire-induced progressive collapse are also discussed.
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NOTATION

All symbols used in this thesis are defined where they first appear. For the reader’s convenience, the principal meanings of the commonly used notations are contained in the list below. The reader is cautioned that some symbols denote more than one quantity; in such cases the meaning should be clear when read in context.

Abbreviations

2D,3D 2-dimension, 3-dimension
ACI American Concrete Institute
AISC American Institute of Steel Construction
ASCE American Society of Civil Engineers
BRE Building Research Establishment
BSI British Standard Institution
CDR capacity-demand ratio
CFD computational fluid dynamics
CSA Council of Standards Australia
DEF Dynamic Effect Factor
DIF dynamic increase factor
DoD Department of Defence
FE finite element
FEMA Federal Emergency Management Agency
FRP fibre-reinforced polymer
GSA General Services Administration
NIST National Institute of Science and Technology
ODPM Office of the Deputy Prime Minister
RLS Robustness Limit State
RFCS Research Fund for Coal and Steel
SDOF  single degree of freedom
SR     slenderness ratio
TDA    Temperature-Dependent Approach
TIA    Temperature-Independent Approach
UDL    uniformly distributed load
WTC    World Trade Center

**Roman Symbols**

- $a$: distance from the face of column to the first shear stud
- $A_c$: effective cross-section of concrete flange ignoring the ribbed part
- $A_{eq}$: equivalent steel section area for a composite beam calculated based on modular ratios
- $A_s$: Beam or reinforcement cross-sectional area
- $b_c$: width of column face
- $B$: width of slab
- $d$: overall depth of slab/concrete flange
- $d_1$: depth of slab ‘cover’
- $d_2$: depth of slab ‘rib’
- $d_b$: bolt diameter
- $d_f$: maximum ductility supply of component
- $d_y$: elastic yielding deformation of component
- $D$: distance between stiffeners
- $D_a$: actual floor deflection subsequent to column buckling, see Eq.(6.1)
- $D_d$: deflection due to sudden column loss after column buckling, see Eq.(6.1)
- $D_s$: idealised full static floor deflection after column buckling, see Eq.(6.1)
- $D_H$: diameter of bolt hole
\( e_{\text{sys}} \) enhancement factor considering the slab membrane action

\( E_s \) elastic modulus of steel

\( E_c \) elastic modulus of column

\( E_{c1} \) initial tangent modulus of concrete in compression

\( E_A \) axial rigidity

\( E_I \) flexural rigidity

\( f_y \) yield strength

\( f_c \) compressive strength of concrete

\( f_{ctm} \) concrete tensile strength

\( f_t \) axial tensile strength of concrete

\( f_u \) ultimate strength

\( F_{Rd} \) resistance of joint component

\( F_{tr,Rd} \) tension resistance of bolt-row \( r \) in a bolted connection

\( g \) gravitational acceleration

\( h_c \) column section depth

\( h_r \) distance from bolt-row \( r \) to the centre of compression

\( H \) ceiling height

\( I \) second moment of area

\( I_c \) second moment of area of column

\( k \) stiffness coefficient of joint component

\( k_I \) stiffness coefficient of shear zone components

\( k_2 \) stiffness coefficient of compression zone components

\( k_c \) coefficient that allows for the self-equilibrating stresses and the stress distribution in the slab prior to cracking

\( k_e \) elastic stiffness of component

\( k_{\text{eff}},r \) effective stiffness coefficient for each bolt-row \( r \) in a bolted connection
$k_{eq}$ equivalent stiffness coefficient for the basic components of all bolt-rows in a bolted connection

$K_a$ axial stiffness of the beam

$K_r$ Stiffness of rotational restraints

$K_l$ Stiffness of axial restraints

$L$ length of structural member

$L_b$ beam length

$L_c$ column length

$L_t$ ‘transmission’ length

$m$ distance between top stiffener and load application point

$m_{plc}$ plastic bending moment of the column face per unit length

$M_{j,Ed}$ design bending moment acting on the joint

$M_{j,Rd}$ design moment resistance of the joint

$M_u$ ultimate restoring moment at test column ends

$N$ axial force

$N_{pl,Rd}$ plastic axial resistance of beam

$P_{\text{total}}$ total gravity load applied on all the floors above the fire affected column

$P_i$ suddenly applied gravity load

$P_i$ maximum load resistance offered by the column under its buckling temperature

$P_0$ initial column axial force under ambient condition

$P_{\text{gravity}}$ overall gravity loading applied on the upper ambient double-span floors

$P_t$ axial force of column during fire

$Q$ rate of the heat release

$r$ radial location from the flame axis

$r_c$ normalised residual compressive strength

$R_{b,rd}$ resistance for plate in bearing
$s$ distance from reinforcement to slab top

$s_c$ normalised compressive strength starting compressive nonlinearity

$S_j$ secant joint rotational stiffness corresponding to the design moment $M_{j,Ed}$

$S_{j,ini}$ initial joint rotational stiffness

$t$ thickness of the steel deck

$t_c$ thickness of column face

$T_{max}$ maximum temperature

$T_r$ ceiling temperature at distance $r$

$u_o$ initial column top position, see Fig. 8.18

$u_n$ new shifted coordinate origin, see Fig. 8.18

$u_{max}$ maximum deformation capacity of component

$u_d$ dynamic deformation of the floor system

$u_s$ static deformation of the floor system

$U_{c,i}$ strain energy dissipated by buckled column under $P_i$

$U_{f,i}$ strain energy dissipated by floor systems under $P_i$

$w_1$ width of slab rib bottom

$w_2$ width of slab rib top

$w_q$ deflection caused by the design load

$W_i$ external work done by suddenly applied gravity load $P_i$

$W_{support}$ work done by support force during support removal procedure, see Eq.(8.5)

$W_b$ internal work done by the beams in bending

$W_e$ external work done per unit load

$W_s$ internal work done by the composite slab in bending

$z_0$ vertical distance between the centroid of the uncracked, unreinforced concrete flange and the uncracked, unreinforced composite section

$z_{eq}$ equivalent lever arm
Greek Symbols

\( \alpha \) non-dimensional weighting factor based on load distribution on the floor

\( \beta_r \) correlation factor for sudden column loss

\( \beta_t \) coefficient in Eq. (4.19) taken as 0.4 for short-term loading

\( \gamma_{M,fi} \) reduction factor for material

\( \delta \) coefficient in Eq. (4.19) taken as 0.8 for high ductility bars

\( \delta u_s \) increment of static deflection

\( \delta U \) increment of absorbed strain energy

\( \delta U_{f,j} \) incremental internal energy absorbed by one individual floor system \( j \)

\( \delta W \) increment of external work

\( \varepsilon \) strain

\( \varepsilon_c \) compressive strain in concrete at \( f_c \)

\( \varepsilon_{smu} \) ultimate mean strain of embedded reinforcement

\( \varepsilon_{t1} \) tensile strain in concrete at \( f_t \)

\( \varepsilon_{t2} \) ultimate tensile strain of concrete

\( \theta_u \) end rotation of test column

\( \mu_1 \) strain hardening coefficient after yielding

\( \mu_2 \) further strain hardening coefficient after ultimate strength

\( \mu_u \) friction coefficient

\( \nu \) Poisson’s ratio

\( \rho \) longitudinal reinforcement ratio in composite beam

\( \Delta_{sr} \) increase of reinforcement strain when the first crack forms

\( \Delta_{u,s} \) maximum reinforcement elongation

\( \Delta T \) temperature increase

\( \Pi_{int} \) total internal work

\( \sigma \) stress
\( \sigma_{r1} \) reinforcement stress at the location of first crack

\( \tau_{sm} \) average bond stress along the transmission length

\( \varphi \) ductility index of component

\( \Phi \) diameter of reinforcement bar
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1.1 Background

With the development of fabrication facilities, architectural concepts, and construction skills, steel has become one of the most popular materials for construction, particularly for stadiums, exhibition halls, airports, industrial buildings, and tall buildings. According to recent statistics, 79% of tall buildings over 200m height are steel structures based on an investigation in 1990 (Hu, 2007). The wide use of steel is attributed to its relatively light weight, high strength, good ductility and plasticity, flexible structural layout, and quick fabrication potential. On the other hand, although the development of architectural concepts has led to modern buildings that are relatively light, flexible and capable of providing longer spans, these structures may be more vulnerable to loading conditions beyond the normal design criteria anticipated in daily operation, such as blast, impact, and fire. In particular, steel has poor fire-resistance which cannot be ignored in structural design. Under fire conditions the strength and stiffness of steel decrease rapidly with the elevation of temperature. A fire developing in an unprotected building can affect a wide floor area, and this may subsequently result in a total collapse of the structure. Even a rather localised fire which develops in an unprotected steel/composite structure can lead to a severe heating of the nearby structural elements (connections, beams and columns). This may result locally in a significant reduction of the carrying capacity of one or two columns and subsequently to a loss of global stability of the structure. The corresponding failure mechanism can be referred to as ‘fire-induced progressive collapse’.
Public awareness of building safety against instability and collapse has significantly risen during the past few decades. More than forty years ago, structural progressive collapse triggered by local key component failures started to be studied by structural engineers. The collapse incident of the London Ronan Point apartment block in 1968 initiated a worldwide concern on this collapse mechanism. The United Kingdom was the first country to consider progressive collapse in its design codes and regulations. More recently, the design requirement against progressive collapse started to be introduced in the codes of the United States and some other developed countries. The World Trade Centre terrorist attack in 2001 highlighted the importance of structural robustness design against progressive collapse, and boosted a global attention on this issue. Although there have been fewer records of massive progressive collapse directly caused by fires only (i.e. in the absence of other extreme load combinations), it cannot be guaranteed that existing conventional fire design methods and procedures are adequate to offer an integrated solution of safety measures to limit the potential for fire induced progressive collapse. This is largely due to the lack of the knowledge of structural response subject to key member loss scenarios at elevated temperatures.

1.2 General strategies for progressive collapse prevention

The ability of a structure subject to extreme loads to resist the progression of local damage to global failure is referred to as ‘robustness’. After the incident of Ronan Point in 1968, significant efforts were devoted to establishing design concepts against such type of massive structure collapse, and since then related approaches were widely introduced among engineers and researchers (Burnett, 1975; Breen and Siess, 1979; GSA, 2003; Izzuddin et al., 2008; Vlassis et al., 2008; DoD, 2009). Widely accepted strategies for mitigating progressive collapse include the improvement of ductility, redundancy, local resistance and continuity of the structures (Izzuddin et al., 2008). Through intensive investigations on the progressive collapse mechanism, which include experimental investigations and numerical modelling approaches, these general robustness enhancement strategies (either prescriptive or performance-based) are believed to be effective, although they are normally considered under ambient conditions while fire effects are not considered from a robustness perspective. Considering the effectiveness of structural design, the essence of the robustness requirement against progressive collapse should be adequately utilising the widely
accepted strategies (ductility, redundancy, continuity, and local member resistance), and should ensure that the load can be effectively redistributed without causing further collapse. In most cases the four aspects are interdependent and cannot be considered separately in structural design solutions.

Sufficient ductility provision requires that as the members and joints develop plasticity, increased deformation capacity can dissipate more energy and can facilitate the load redistribution process. As a result, the stability of the damaged structure can be maintained with a new equilibrium condition without developing further structural failure, even if at the cost of excessive local deformations. Recent studies concluded that sufficient ductility supply of joints is the most essential aspect to allow the development of catenary/membrane actions to increase the overall structural resistance to progressive collapse (Izzuddin et al., 2008; Vlassis et al., 2008).

Redundancy is related to the limitation of the effect of local failure, and is closely associated with structural ductility. The concept of enhancing structural redundancy is similar to the design of aircrafts that adopt multiple engines – even if one of the engines fails, the aircraft can still operate thanks to the contribution of other engines. Similarly, when a key member is damaged, it is required that adjacent members can sustain additional redistributed loading without failure. For example, when a column is severely damaged due to explosion, the effectiveness of structural redundancy depends on the capability of the surrounding columns and joints resisting the increased load in a double-span manner. Of course, structural ductility should be closely relied upon to maintain the structural stability at large deformations, thus to sustain structural redundancy.

The priority of increasing the structural redundancy is to ensure the provision of continuous alternative load path, which highlights the structural continuity requirements. The tie force method, which demands that the continuity of load path distributed among beams, columns, slabs, joints, and bearing walls should be considered during the structural design procedure, is based on this principle. However, under certain circumstances, good structural ductility, redundancy and continuity can result in negative effects (Izzuddin et al., 2008). This is like a fuse wire in an electrical circuit – the blowing out of the fuse can prevent the electrical circuit from overloading. In this respect, some researchers suggest that a structure can be divided into several strong
parts and weak parts, which may effectively avoid the progression of collapse if the extreme load is applied on weak components (Magnusson, 2004).

Finally, the provision of local resistance can be also beneficial in resisting progressive collapse. Consider a steel/composite multi-storey car park for example, a localized fire in the vicinity of a column may greatly decrease its resistance, and progressive collapse may be initiated due to the failure of the steel column and lack of ductility supply of the upper ambient floors. However, if the affected column is protected (e.g. by light-weight concrete), the development of temperatures in this column can be significantly retarded. At the end of the fire, the protected column may not fail at all, thus avoiding failure of the car park. However, locally enhancing the resistance of key elements is usually not straightforward because the targeted locations are hard to predict under unforeseen events, thus increasing local resistance is potentially expensive and difficult to comprehensively address in design codes.

1.3 Design approaches against progressive collapse

Based on the above strategies, the design approaches for assessing progressive collapse are normally categorized into two types, namely, direct design approach and indirect design approach. Currently, both approaches are widely used, but different focuses are given with respect to their applicability and sophistication.

1.3.1 Direct design approach

The direct design approach is normally performance-based and relies on sophisticated analysis tools. It requires a series of analysis based on failure predictions, e.g. the location of damage, and then it needs the assessment of the robustness of the structure following an extreme loading event that can cause local failure with the potential to initiate progressive collapse. Two sub-methods are usually employed in the direct design approach framework, namely, the alternative path method and the specific load resistance method. Despite its rationality, the direct design approach may not be commonly used in daily design practice due to its potential requirement for dynamic analysis.

The alternative path method focuses on investigating the response of a structure subject to the local failure of a certain member, and it is required that the surrounding structure
can effectively resist and redistribute the load originally taken by the failed component, through which the stability of the structure is maintained and further collapse is prevented. One of the most widely used alternative path methods for robustness assessment is the ‘sudden column loss’ scenario, which assumes an abrupt removal of the targeted column to represent the consequence of extreme loading (e.g. blast). Generally, the analysis procedures for the alternative path method can be selected from the following: linear static analysis, linear dynamic analysis, nonlinear static analysis, and nonlinear dynamic analysis, depending on the design requirements.

Linear static analysis and linear dynamic analysis exclude the inelastic structural deformation and material nonlinearity, and thus tend to offer only a rough robustness evaluation with controversial reliability. Nonlinear static analysis enables the designer to capture the geometric and material nonlinearities which explicitly account for the catenary/membrane action in the floor systems subject to large deformations and the development of material plasticity. However, dynamic effects in nonlinear static analysis are unsafely ignored but can be alternatively considered through either a conventional Load Increase Factor method or a new method named Ductility-Centred Approach recently developed at Imperial College London (Izzuddin et al., 2008, Vlassis et al., 2008). In this new method, simplified dynamic assessment using the principle of energy balance, instead of directly undertaking complex nonlinear dynamic analysis, is utilised employing the obtained nonlinear static response. Currently, the Ductility-Centred Approach is suitable for the sudden loss of a single column, while it is not directly applicable to fire induced column loss scenarios. Nonlinear static analysis can also be effectively used for parametric studies to highlight the influence of different structural properties on progressive collapse resistance. Finally, nonlinear dynamic analysis is the most accurate way to reflect a real structural response, but it normally has to be performed through the aid of sophisticated numerical tools, e.g. finite element analysis packages. Therefore, it is quite computationally expensive hence it is more suitable for research purposes than for routine design practice.

On the other hand, instead of removing critical members as specified in alternative path method, the specific load resistance method is concerned with strengthening the areas that are believed to be more prone to extreme loads. This method encourages the designers to reasonably increase the strengths of key members to resist predictable
threats. With respect to fire hazard, passive fire protection coating is one of the general methods that comply with the specific load resistance method. However, as mentioned above, the location, the type and the scale of extreme loads are usually unpredictable, which often leads to some difficulties in practice.

1.3.2 Indirect design approach

The indirect design approach focuses on increasing the robustness of structures by prescriptively enhancing the continuity, redundancy and ductility. This can be realized by design procedures control, including structural type selection, member size selection, the structural layout arrangement, ductility requirement of the joints, as well as fire protection locations specifically for fire scenarios. Therefore, this approach is prescriptive and event insensitive, and can be applied to different structural systems, i.e. steel/composite structures or concrete structures. One typical method that belongs to the indirect design approach is the ‘Tie Force Method’ (ODPM, 2004), which demands that the tying force provided by members and joints should be sufficient to allow redistribution of gravity loads, though the latter is not explicitly considered. A major shortcoming of this method (ODPM, 2004) is the neglect of ductility issues, making it potentially unsafe for typical building structures that may be subject to sudden column loss (Izzuddin et al., 2008).

1.3.3 Codified treatment

The above design methods are widely adopted in various design codes and recommendations. The US Department of Defence (DoD, 2009) adopted the alternative path method and the tie force method for its direct and indirect design methods respectively. The US General Services Administration (GSA, 2003) suggests that joint details and structural redundancy should be carefully considered during the design procedure, and then the alternative path method should be followed after completing the primary design. The British Standards (BSI, 2001; ODPM, 2004) also requires that sufficient horizontal and vertical ties should be provided.

Furthermore, the level of resistance is suggested to be dependent on the function and occupancy conditions of the considered building. In this respect, the US Department of Defence (DoD, 2009) classifies structures into four categories with different levels of importance, from category I structures which are low occupancy buildings to category
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IV structures including essential facilities and military assets. Design requirements against progressive collapse are varied according to the category to which the structure belongs. For occupancy category I, no specific requirements are needed. For occupancy category II, either the tie force method combined with the enhanced local resistance method or the alternative path method can be employed. For category III occupancy, the alternative path method is required with the enhanced local resistance method for all perimeter first storey columns or walls. The highest category, category IV, requires that tie forces, alternative path and enhanced local resistance methods are used on specific members.

1.4 Fire induced progressive collapse

Two types of fire hazards are mainly considered in current fire resistance design guidance, namely, fully-engulfed fire and localised fire. For a fully-engulfed fire, the affected area is usually massive, and it may be conservatively assumed that a uniform fire loading is applied on all structural members in the fire compartment. A fully-engulfed fire is highly likely to cause global instability of structures if the temperature is sufficiently high, and the collapse mode or the locations triggering total collapse is normally varied and complex. Therefore, failure characteristics of structures under a fully-engulfed fire are relatively large scale, and do not feature a typical progressive collapse mode which is usually related to local loss of one or two members with adjacent members unaffected. On the other hand, a fire in a building probably remains localised provided that the compartment or the opening is sufficiently large. A localised fire has its unique thermal characteristic which is different from a fully-engulfed fire. The main differences are that for a fully-engulfed fire, the whole compartment is filled with smoke, combustion products and air, all fire loads are burning such that the gas temperature all over the compartment can be seen as uniform. While for a localised fire, only a limited part of a compartment is in fire, the temperature in the plume and the surrounding gas are not uniform, thus these need to be considered separately. A typical localised fire scenario is associated with burning of one or more cars in a car park, which may severely affect the nearby steel column and floor system but leave the much cooler surrounding substructures unaffected. Also, some building types (e.g. atriums) have large spacing that can avoid a full development of fire. Regarding the possibility of progressive collapse, a localised fire which develops in an unprotected steel/composite
structure can considerably deteriorate the resistance of certain key members (connections, beams and columns) within the heating area. As a result, a significant reduction of the carrying capacity of the fire affected column as well as the adjacent joints and floor system can potentially lead to a loss of global stability of the structure, hence inducing progressive collapse. Compared to structural collapse induced by fully-engulfed fires, initial failure caused by a localised fire is more predictable, e.g. column buckling. In this thesis, focus is only given to structural robustness against localised fire.

While to some extent fire induced progressive collapse features a common characteristic to blast and impact induced progressive collapse, which normally involve structural resistance against the progression of local damage towards global failure, structural robustness under fire conditions has its own unique resistance mechanism, thus current progressive collapse design recommendations are not directly applicable. Considering the alternative path method used for progressive collapse assessment under ambient conditions for example, it is assumed that a key member (usually a column) is suddenly removed, and then the ductility, strength and stability of the remaining structure are assessed. On the other hand, when a structure is subject to a localised fire near the same column, the column will expand first at lower temperatures due to the thermal expansion. Even if the ‘critical temperature’ (the temperature at which the column starts to buckle) is reached, the fire affected column may still have considerable residual resistance and is capable of resisting the applied load in conjunction with the upper unaffected floors. Besides, compared to sudden column loss scenarios, dynamic effects may not be that significant during a fire. Furthermore, thermal effects such as thermal expansion, thermal bowing, and material deterioration can considerably complicate the prediction of structural behaviour, and these are not yet considered in current guidelines dealing with progressive collapse. Finally, even for a geometrically regular building, different behaviours are expected between the affected floor and the upper ambient floor. A likely situation is that the fire floor system is not capable of withstanding the applied load in a double span condition after the column beneath is damaged, while for the upper ambient floors which could be stronger, each floor has to share the additional load from the fire affected floor apart from resisting their own loads in double span. This mechanism is different from the idealised column loss scenario specified in existing design guidance and codes. Figure 1.1 illustrates the progressive collapse of a 2D steel frame subject to localised fire, where the fire origin is assumed to occur at floor level 3.
near the right internal column, and the fire affected members (column and two adjacent beams) are assumed to have a uniform temperature distribution across the cross-section and along the length with the same temperature elevation rate.

Fig. 1.1 Deflection shape of 2D steel frame subject to localised fire: a) Deflection shape after column buckling, b) Total collapse of frame

In terms of possible design solutions against fire induced progressive collapse, the effectiveness of water sprinklers may evidently be contemplated, but such a solution increases the construction costs and requires constant maintenance. Moreover, a severe fire may affect the proper functioning of the sprinklers. Applying fire protection coatings is also a possible solution, but the influence of costly fire protections on the progressive collapse resistance of structures has not been fully understood. Towards an economical design for providing the structure with sufficient robustness, the structure should be alternatively designed in such a way that an unforeseen fire does not lead to progressive collapse in the absence of passive fire protections and sprinkler systems. In other words, while the structure is designed in the usual way under normal loading conditions, it should be able to undergo a complete or partial loss of a structural element without losing its global stability even at the cost of excessive deformations or displacements, thus preventing progressive collapse. In current codified documents, e.g. Eurocodes and British Standards, such a performance-based approach is allowed but no specific design recommendations or even basic application principles are provided.

As a general remark, a wide knowledge gap still exists for the consideration of fire hazard in design-oriented progressive collapse assessment, while directly applying existing progressive collapse assessment methods to the fire scenarios is not appropriate.
Of course, these methods can be drawn on, but significant modifications are necessary to adapt to the specific case of fire. In this context, additional considerations exclusively related to fire-induced progressive collapse have to be addressed, including:

1) *Influence of thermal effects on structural robustness.* Due to the divergence of the fire development process in different structure types, the scale and the severity of a fire is hardly predictable. Considering a localised fire that occurs near an internal column, it may be acceptable to assume an even temperature distribution within the column, but the temperature will vary significantly over the adjacent beam lengths. In addition, the temperature distribution across the cross-section of the steel beams and concrete slab will be nonlinear, and may have to be simulated by high-order nonlinear functions if certain accuracy is required. In addition, thermal expansion can induce significant axial forces in beams, which will to some extent change the behaviour of joints, e.g. increasing joint rigidity through ‘pre-compression’. The influence of these thermal effects on structural robustness should be extensively evaluated, because some can be significant while others may be negligible. Typically, conducting thermal transfer analysis for each type of structure is the most accurate solution, but it is cumbersome for design-oriented practice; therefore, idealised and simplified event-independent temperature profiles are required and expected to be directly applied onto the column and the adjacent areas of beams and slabs with favourable reliability. With this objective, relevant information from typical fire events and experiments need to be collected, and parametric studies are required to verify the reliability of any developed empirical or semi-empirical simplified thermal load cases.

2) *Ductility supply of joints under both ambient and fire conditions.* A sufficient ductility supply of joints is the most essential aspect to maintain the stability of damaged structures allowing the attainment of a new equilibrium configuration without developing further structural failure. When a column subject to fire partially or totally loses its load carrying capacity, the ambient part of the structure tends to prevent the structure from collapse with its own resistance. This resistance of the undamaged part of the structure largely depends on the ‘ductility supply’ from the joints. In other words, the failure criteria could be defined according to whether the ductility limits of the joints are exceeded. Due to the highly nonlinear response of the
structure in fire, e.g. catenary action of beams, a joint is likely to be subjected to considerable moment, axial load and shear force at the same time, thus the M-N-V interaction characteristic of a joint is rather important. In view of this, the reliability of the joint behaviour modelling is a key factor that determines whether the robustness of a structure can be properly evaluated. In this context, the Ductility-Centred Approach for ambient sudden column loss scenarios (Izzuddin et al. 2008; Vlassis et al. 2008) can be drawn on for this study with appropriate modifications to establish the maximum ductility demand and its comparison against the ductility supply under either ambient or elevated temperature conditions. Normally, two modelling approaches for joint modelling can be employed. The first approach is named ‘sophisticated joint model’, of which the main aim is to predict the M-N-V response or moment-rotation characteristics of joints as accurately as possible, where nonlinear shell or solid finite elements would be used. The second approach is called the ‘component-based spring model’ that is similar to the ‘component method’ recommended in Eurocode 3 (2005a). In this approach, bolt-rows or contacting positions are represented by axial springs, and the load-displacement behaviour of the spring is obtained by analysing each component with a possible consideration of group effects. The advantage of the spring model is that the spring assemblies can be directly put into a 3D frame model with sufficient accuracy. With respect to the prediction of the joint behaviour at elevated temperatures, a ‘temperature reduction factor approach’ can be employed for the component-based joint model, where the reduction factors for strength and stiffness for the components are required.

3) Undesirable failure mode of fire affected joints. When a large deformation is induced due to key member removal, joints are likely to be subjected to a combination of bending moment, axial load and shear force simultaneously. Indeed, even if considerable burdens are exerted on far-end joints in a dynamic manner, shear failure is less likely to occur if the surrounding joints are under ambient conditions and if they are designed with sufficient shear resistance reserve to consider the increased shear force due to column loss. Under fire conditions, however, the shear resistance of fire affected joints can be greatly impaired at elevated temperatures; therefore, the shear failure of the fire affected floor has to be carefully evaluated. A likely scenario of joint shear failure under a localised fire is punching shear when the overall shear resistance of the fire affected joint drops below the transferred shear force before
column buckling. This type of shear failure could also occur after column buckling, where the fire affected floor is ‘pulled up’ by stronger upper ambient floors. It is worth noting that this type of punching shear failure is not likely to occur in typical ambient sudden column loss scenarios.

4) **Benefits of 2D composite slab responses.** Concrete slabs play an important role in the load carrying capacity of a composite structural system. Conclusions from recent research (A-H Eldib, *et al.*, 2009; Bailey and Toh, 2007) indicate that the resistance of a composite floor system is usually underestimated when a ‘grillage modelling approach’ is adopted. This modelling approach takes the ‘effective width’ of the concrete slab and thus may underestimate the load resistance due to an inappropriate consideration of the 2D slab membrane action that may effectively carry extensive loads in double span along two directions. Difficulties of analysing composite slabs are usually caused by their complex geometrically orthotropic properties under fire conditions. In ADAPTIC (Izzuddin, 1991), which is a nonlinear finite element analysis program used to numerically verify the various methodologies presented in this thesis, a new shell element for the realistic modelling of composite and reinforced concrete floor slabs (considering the geometric orthotropy of ribbed slab) subject to extreme loading conditions was developed by Izzuddin *et al.* (2004). This element will be applied for simulating the composite slabs in this study. Extensive validation of this slab model against experimental results on flat and ribbed reinforced concrete and composite floor slabs has been conducted by Elghazouli and Izzuddin (2004a), and good correlation between the numerical predictions and experimental results was demonstrated.

5) **Potential dynamic effect under fire conditions.** It has been long recognised that the dynamic effect associated with idealised sudden column loss scenarios in the alternative path method plays an important role in structural robustness assessment procedures, and thus this effect needs to be carefully evaluated. The conventional design-oriented solution for considering dynamic effects in progressive collapse assessment is the so-called load factor approach, where a dynamic increase factor (DIF) is multiplied with the static vertical load to obtain the increased static response, though the selection of DIF is typically fraught with inaccuracy. Alternatively, detailed nonlinear dynamic analysis may be employed for a more accurate
assessment, though this can be prohibitively computationally expensive. In this context, the Ductility-Centred Approach has been developed to address the aforementioned shortcomings, as it offers the convenience of conducting simplified dynamic assessment using the principle of energy balance instead of directly undertaking detailed nonlinear dynamic analysis. However, all the above methods considering dynamic effects are based on sudden column loss assumptions, whereas under localised fire conditions where columns are directly exposed to fire, potential dynamic effect may be much less significant due to potential residual resistance of the fire affected column during buckling. In other words, assuming a sudden loss of fire affected column is potentially too conservative. Contrarily, assuming column buckling to be a fully static phenomenon may underestimate the ductility demand in the supported floors, thus yielding an unsafe result. Therefore, an appropriate evaluation of the dynamic effect caused by column buckling during a localised fire is essential for predicting a reasonably accurate ductility demand of a structure.

6) **Suitable behaviour models for robustness assessment under fire.** For typical ambient progressive collapse analysis where a sudden column loss scenario is proposed, the behaviour/sub-structural model may include the affected floors above the removed column with appropriate boundary conditions to represent the interaction with the surrounding affected structure. For a regular building where the affected floors are identical in terms of structural layout and loading, investigating a reduced model consisting of a single floor system is usually sufficient (Izzuddin et al., 2008). Under localised fire conditions, however, considerable differences are anticipated between the behaviours of the fire affected floor and the ambient floors, even if the considered structure is regular in terms of structural layout and loading. Therefore, a behaviour model considering localised fire should be able to capture the behaviour of both fire and ambient floors, as well as the influence of surrounding cool sub-structures. Inspired by the multi-level modelling approach developed in the Ductility-Centred Approach for progressive collapse assessment under ambient sudden loss scenarios, the development of modified multi-level behaviour models considering localised fire is expected in this study. The reliability of each level of model needs to be checked through undertaking comparisons among the models as well as parametric studies. The rationality of the frame behaviour model also depends on the accuracy of boundary condition simulations which have to be evaluated carefully. Finally,
recommendations for the FEM simulations of structures under localised fire are desirable.

7) Derivation of performance-based design recommendations. This is an important objective for the study. Developing performance-based design recommendations is in principle similar to the alternative path method adopted in the direct design approach for robustness, but additionally requires the consideration of fire. Firstly, similar to the accepted column loss scenario representing the effect of blast, a reasonable fire scenario needs to be proposed and conveniently applied to structures. Subsequently, by incorporating the fire scenario and analysing the multi-level substructure model, the resistance and ductility requirements in the damaged frame can be evaluated. Through comparing these demands to the available ductility supply that the frame components provide, robustness requirements will be derived. Furthermore, different fire locations need to be considered, since the structural responses and the robustness requirements for different fire locations may differ significantly.

8) Derivation of practical recommendations for routine design. When a complex performance-based design approach is employed, for each particular structure being designed, the specific requirements applicable to the structure and its scenarios have to be expressed before the check of the structural robustness. For example, a thermal load case should be applied to a structure, and the resistance of each member will be assessed in accordance with this fire scenario. This performance-based approach is relatively accurate, but it requires significant computational and modelling efforts. On the other hand, the indirect design approach focuses on adopting general structural integrity measures throughout the design process without the need for additional analysis. This brings convenience to daily design practice, or at least efficiency for the preliminary design stage. This idea could be drawn on for robustness assessment under fire situations. For example, instead of checking the ambient or elevated temperature ductility demand/capacity of the floor system for a specific structure, a practical design recommendation on how to select the joint detailing (i.e. in the first design steps), which structural components need to be strengthened, or where to apply fire protection could be very helpful and can effectively ensure efficient structural design in the case of localised fire before a more detailed structural fire assessment is performed.
1.5 Scope and organization of the thesis

According to the above outlines which form the basis of the objectives of this study, the main framework of this thesis involves both simple and sophisticated design-oriented approaches for the robustness assessment of steel/composite buildings subject to localised fire. In this context, focus is given to fire loading simplification, fire resistance of individual members, structural modelling idealisation, structural ductility supply assessment, and robustness design efficiency. The nonlinear structural analysis program ADAPTIC (Izzuddin, 1991) is utilised in this thesis to model the reference structures subject to localised fire with a comprehensive consideration of both geometric and material nonlinearity under elevated temperature conditions.

In Chapter 2, the current literature with respect to both fire engineering and progressive collapse is comprehensively reviewed. For the fire aspect, the design approaches ranging from simple prescriptive approaches to more sophisticated performance-based approaches are discussed, and particular attention is given to the design process and the relevant research on steel columns and floor systems, which play a key role in the study of structural response under localised fire. Moreover, fire tests performed on structures/sub-structures during the last few decades are introduced. This is followed by a discussion of recent investigations on progressive collapse, including recognised progressive collapse types, newly developed guidelines, theoretical assessments, experiments, and numerical studies. Finally, gaps between the two aspects in current design practice and research are identified.

The thesis proceeds by investigating the fire response of individual structural members in Chapter 3. Typical fire tests on steel/composite beams, steel columns, composite slabs, as well as the corresponding numerical studies conducted by other researchers are referred to. These tests and numerical simulations are used for benchmark studies verifying the accuracy of the simulation results predicted by ADAPTIC, and thus gaining confidence to extend the individual member modelling to more sophisticated structure/substructure modelling. This is followed by parametric studies to further explore the behaviour of each individual structural member with different material properties, loading conditions (mechanic or thermal), and boundary conditions.
Chapter 4 discusses the response of joints and ductility supply of structures under extreme loading. The component method for joint modelling under ambient conditions recommended in Eurocode 3 (2005a) is introduced first, and an extension of this method to the consideration of elevated temperature conditions is then presented. The chapter proceeds with establishing a multi-failure criterion to estimate the available joint ductility supply, which is used later in the robustness assessment framework developed in this study to determine the vulnerability of progressive collapse through comparison to ductility demands. Finally, illustrative examples of the component-based joint models are presented, and the reliability of the proposed component-based model in predicting the joint responses under both ambient and elevated temperature conditions is validated through comparisons to various available joint tests, including ambient and elevated temperature tests on flush end-plate joints, flexible end-plate joints, and double web-cleat joints.

With sufficient confidence established in the modelling of individual structural components, e.g. beams, columns, floor systems, and joints, Chapter 5 performs a detailed numerical study on a reference structure subject to a proposed localised fire scenario. The main aim of this study is to capture the fire behaviour of the structure as accurately as possible through employing a real temperature distribution obtained from a detailed heat transfer analysis. Firstly, a multi-storey composite car park, designed as a reference building, in accordance with typical current European practice (Gens, 2010) is considered. Subsequently, a multi-level modelling approach is proposed for assessing the robustness of multi-storey buildings subject to localised fire. The effect of each idealisation procedure on the modelling accuracy is carefully evaluated, and two levels of structural modelling are finally selected to simulate the reference car park subject to fire that is assumed to occur at various floor levels. Finally, an elaboration of the obtained structural response subject to fire at each considered floor level is provided, where the joint ductility supply and joint failure criteria are utilised to assess the robustness of the structure. The approach used in this chapter is named as the detailed Temperature Dependent Approach (TDA).

Parametric studies are conducted in Chapter 6 in order to further investigate the influence of various thermal characteristics on the performance of the reference building subject to the selected fire scenario. The main purpose of the parametric study is to
develop an idealised temperature field within the structural model towards a simplified Temperature Dependent Approach (TDA). This is because, although heat transfer analysis in detailed TDA can be established to assess the vulnerability of a structure to progressive collapse under well-defined fire scenarios, this approach needs significant computational and modelling efforts. Hence, it is too complicated for design-oriented application, and does not comply with typical robustness provisions, which are intended to limit the progression of local damage under unforeseen events. To avoid a complex heat transfer analysis and the subsequent structural analysis with a large number of thermal input parameters, an alternative elevated temperature analysis with simplified and idealised temperature distributions is developed to provide a reliable yet much quicker robustness assessment. An important advantage of the simplified TDA will be its avoidance of complex heat transfer analysis, which allows the elevated temperature analysis of structures being performed over a monotonically increased temperature domain in a performance-based manner instead of over a more complex conventional time domain. The reliability of the simplified TDA is assessed through comparison against the results from the detailed TDA. Afterwards, illustrative examples employing the simplified TDA are provided, and a quantitative robustness assessment index, Capacity-Demand Ratio, is presented.

Employing the framework of simplified TDA developed in Chapter 6, Chapter 7 compares the influence of different structural design schemes (e.g. different slab properties, different fire protection schemes, and different joint details) on the robustness of the reference structure, bearing the main aim of deducing a practical robustness design recommendation for typical steel-composite structural buildings against progressive collapse under localised fire. Towards an indirect design approach, this chapter focuses more on the derivation of well-defined design and retrofitting guidelines adapted to industrial requests for design efficiency, which is so-called a “prescriptive” criterion suitable for daily design practice or at least for the preliminary design stage.

In Chapter 8, overcoming the complexity of performing elevated temperature analysis in either detailed or simplified TDA analysis, a Temperature Independent Approach (TIA) is proposed as a further simplified alternative. This alternative approach is event insensitive, in the sense that the details of fire are assumed to be unknown, which is
closer to typical robustness assessment provisions where event-independent local damage scenarios are stipulated. The development of TIA is inspired by the idea of ‘sudden column loss’ scenarios typically adopted in some of the progressive collapse assessment guidelines considering blast effects (GSA, 2003; DoD, 2009), and is an extension of the multi-level Ductility-Centred Approach previously developed at Imperial College London (Izzuddin et al., 2008; Vlassis et al., 2008). TIA models do not require thermal analysis and can thus provide a simplified robustness assessment procedure regardless of temperature. Firstly, the principle of the multi-level Ductility-Centred Approach is recalled. This is followed by a thorough elaboration of the modified Ductility-Centred Method (i.e. TIA) involving additional factors exclusively associated with localised fire scenarios, e.g., equivalent fire affected member removals, and additional strain energy absorbed by the fire affected column during buckling. Afterwards, examples illustrating the application of the new TIA models are provided, where the reference car park considered for the preceding TDA analysis is employed but is alternatively treated using the TIA robustness assessment procedures.

Finally, important conclusions derived from this work are summarized in Chapter 9. Recognising some potential deficiencies of this research, further work in this area is also suggested.
CHAPTER 2

Literature Review

2.1 Introduction

Fire hazard can be a key factor that triggers progressive collapse because elevated temperatures have tremendous effects on construction materials such as steel and concrete. With increasing temperature, the elastic modulus and strength of construction materials can decrease dramatically. Major codes (CSA, 1998; BSI, 2003; Eurocode 3, 2005b; AISC, 2005) have detailed descriptions of the behaviour of steel under elevated temperatures. With regard to concrete, the material properties under high temperatures depend on the constituents, the duration under fire condition as well as the temperature elevation rate. These are fully introduced in major concrete codes (Eurocode 4, 2005; ACI, 2005). Due to the degraded properties of steel and concrete, the robustness of structures subject to fire can significantly deteriorate.

The term resistance to fire induced progressive collapse is literally comprised of two inherent resisting mechanisms, namely, fire resistance and progressive collapse resistance. Robustness behaviour of structures and fire behaviour of structures are two important subjects, thus understanding the two separate topics comprehensively constitutes a major step towards treating both subjects simultaneously. This chapter summarises significant achievements in progressive collapse design (Section 2.2) and structural fire design (Section 2.3) during the last few decades, and concludes with identifying knowledge gaps in the consideration of the fire hazard in design-oriented progressive collapse assessment.
2.2 Progressive collapse design and assessment

Following the gas explosion accident at the Ronan Point apartment block in the early 70’s, when concern was initially developed that local damage of structures could result in progressive collapse, various design requirements were proposed for ensuring sufficient structural robustness (McGuire 1974; Burnett, 1975; Breen and Siess, 1979). The Building Regulations for England and Wales (ODPM, 2004) stipulated the first indirect guideline to avoid the progressive collapse of classified buildings. These design recommendations, including the tie-force approach, are prescriptive and have a limited scientific basis (Izzuddin et al. 2008; Wang and Wald, 2008). More recently, several specialist design guidance (DoD, 2009; GSA, 2003) have began to recognise the fact that sudden column loss scenarios may be more suitable for evaluating structural robustness. The dynamic effects are considered by either defining a load amplification factor or conducting detailed nonlinear dynamic analysis. This is followed by the evaluation of the resistance of each component. If the strength/ductility of any key component is proved to be unable to sustain the applied load, redesign of these weak components may be considered and the whole structure is reassessed. This design process forms the basis of the direct design approach in the newly developed specialist design guidance for structural robustness. Apart from the relevant design guidelines and codes that are serving the engineering society reasonably well, intensive research work on structural robustness has preceded these codes and continues to be undertaken in parallel. These studies are reviewed hereafter.

2.2.1 Typology of progressive collapse

The inducements that lead to progressive collapse can be variable due to different triggering modes and various failure progression patterns. Accordingly, different treatments have to be developed in order to fulfil multiple progressive collapse mitigation requirements. To address this, a typology of progressive collapse was developed by Starossek (2007), where five main types of progressive collapse were identified, namely, pancake-type, zipper-type, domino-type, section-type, and instability-type collapse.
**Pancake-type.** The pancake-type progressive collapse is usually triggered by initial failure of vertical load bearing members (e.g. columns, and bearing walls). Subsequently, a part or a complete floor system above the damaged columns falls towards the lower floor in a ‘rigid-body’ impacting manner, during which the potential energy is transferred into the kinetic energy. Due to the dynamic impact from the upper falling floors, further damage is induced in the lower floor columns, and subsequent failure of the lower floor is triggered. Finally, the whole structure can collapse floor by floor with accumulated upper floor weights. A typical pancake-type failure is the collapse of WTC towers, where the lower floors collapse vertically one by one due to the dynamic impacting of the whole block above.

**Zipper-type.** This progressive collapse type occurs during the loading redistribution process where the alternative load paths following the failure of one or several elements are not adequate. When the surrounding elements with similar functions are not able to resist the redistributed load, further collapse is triggered. Opposite to the pancake-type, the direction of the zipper-typed collapse progression is usually perpendicular to the principle loading resisting directions of the damaged elements. Zipper-type progressive collapses are usually found in bridges. For example, rupture of one cable in a cable-stayed bridge can overload the adjacent cables, thus triggering successive failure of other cables. A similar condition can be found in a continuous beam bridge or a building, where progressive collapse is triggered by the failure of one pier/column which greatly overload the neighbouring piers/columns.

**Domino-type.** Just as the name implies, domino-type progressive collapse is caused by the over-turning of one element that induces a chain reaction consisting of the over-turning of an adjacent element, and so on. This collapse type is not commonly found in normal buildings, but is of major concern for a row of towers (e.g. temporary scaffolding towers, and towers with transmission lines overhead).

**Section-type.** This type of progressive collapse is not typical for an entire structure, but is focused on individual members. Take a concrete beam under bending moment for example, when part of the cross section is damaged (e.g. cracking), the internal stresses resisted by that part are redistributed into the remaining parts of this cross section. The corresponding increase in stress over the remaining parts of the cross-section can cause a progression of failure throughout the entire cross section.
Instability-type. Instability of structures can be efficiently avoided with the use of bracings or other sway resistant structural systems. However, failure of bracing elements can make a system unstable, and could thus lead to progressive collapse. Take a steel framed structure for example, when a compressive bracing member buckles due to overloading, the local part of the structure may experience instability by a small disturbance, which can trigger progressive collapse. Another likely situation is the failure of a column web in compression as a result of contact with lower flange of a steel beam. This can lead to local buckling, overall instability of the affected column, and consequently global collapse.

It should be noted that the five progressive collapse types can interact with each other with various combinations and degrees. Therefore a sixth category named mixed-type collapse was also proposed. Based on potential common characteristics among the collapse types, a higher level of classification was further developed by Starossek (2007), namely, impact class, redistribution class, and instability class. The developed topology of progressive collapse is summarized in Table 2.1.

<table>
<thead>
<tr>
<th>Class</th>
<th>Type</th>
<th>Typical examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Impact class</td>
<td>Pancake-type</td>
<td>Dynamic impacting from upper falling floors</td>
</tr>
<tr>
<td></td>
<td>Domino-type</td>
<td>Over-turning of one transmission line tower in a tower row</td>
</tr>
<tr>
<td>Redistribution class</td>
<td>Zipper-type</td>
<td>Rupture of one cable in a cable-stayed bridge cause further ruptures of adjacent cables</td>
</tr>
<tr>
<td></td>
<td>Section-type</td>
<td>Cracking of concrete beams leads to an overall failure of the beam</td>
</tr>
<tr>
<td>Instability class</td>
<td>Instability-type</td>
<td>Disabled bracing members trigger a sudden instability of a structure</td>
</tr>
</tbody>
</table>

Recognising the five basic progressive collapse types, and considering the current study that is focused in the robustness assessment of steel-composite structures subject to localised fire, four types among the five can be involved in this study, which are pancake-type, zipper-type, section-type, and instability-type. Considering a car-burning accident occurring in the vicinity of a column, the buckling of the fire affected column at high temperatures will redistribute the vertical load to adjacent columns, and three subsequent possibilities are expected:
1) If the adjacent ambient columns are not able to resist the additional redistributed load, successive buckling of these columns will occur, and this collapse mode falls into the zipper-type category.

2) If all the fire unaffected surrounding columns have sufficient load resistance after a complete loss of the fire affected column, but the fire affected ‘double-span’ floor system overhead together with the upper ambient ‘double-span’ floors (if there is any) cannot withstand the vertical load on their own due to poor ductility supplies offered by the joints, these floors will fall down as debris impacting the lower floors. Under this circumstance, pancake-type progressive collapse can happen.

3) If the surrounding fire unaffected columns have sufficient resistance, and the upper ambient floors can withstand the redistributed load without failure, the fire affected floor can be seen as a ‘single-span’ system without column failure due to the pulling up effect of upper ambient floors. However, if the temperature increases to a sufficiently high temperature, local hogging joint failure may occur, and a progressive ‘single-span’ failure of the fire affected beam may be expected. This belongs to the section-type of progressive collapse.

Apart from the three failure types, instability-type collapse may also be possible if the fire occurs near a bracing system, though this collapse type is beyond the scope of this thesis. The classification not only encourages the development of the conceptual treatment of progressive collapse but also potentially helps to develop different levels of design requirements or failure criteria as well as the corresponding countermeasures. Until now, suggestions on defining the ‘Robustness Limit State’ are still insufficient and controversial. Izzuddin (2009) indicated that ‘allowing collapse of the upper floor(s) for the robustness limit state is unrealistic, at least for typical construction’; this conclusion was mainly based on the study undertaken by Vlassis et al. (2009), who showed that even the falling down of only one floor debris can lead to very serious damage on the lower floors. According to this conclusion, the failure of upper floors which potentially triggers pancake-type progressive collapse should not be allowed. On the other hand, Paragraph A3 of Schedule 1 to the Building Regulations 2000 (ODPM, 2004) allows the area at risk of collapse limited to 15% of the floor area of the affected storey or 70m², whichever is the less, but the extension of further damage towards adjacent stories is not permitted. This statement actually ‘relaxes’ the requirement level of the
robustness limit state, but it is not accompanied with specific implementation guidelines, nor is there a recognition of the need to account for debris loading and the great difficulties this entails given the dynamic nature of debris impact. A more detailed discussion on the definition of the robustness limit state and the failure criteria for the current study is presented in Chapters 4 and 5.

2.2.2 Recent research on progressive collapse

A great effort has been devoted to the investigation of progressive collapse during the past few decades, and the progress was particularly fast during the last ten years. Astaneh-Asl et al. (2002) conducted three static tests on simply supported composite floor slabs subject to a side (façade) column loss. The first test was conducted on a normal designed structure while the other two tests were considered on new retrofit cable-based structures. The objective of the tests was to develop a cable-based mechanism that can be used to prevent progressive collapse, by adding cables along façade beams near the flanges, as shown in Figure 2.1. Through comparisons of the outcomes, it was observed that the progressive collapse resistance can be improved by the tensile resistance offered by the additional cables. Even if the beam-to-column connections were to fail, the cables could still provide a sufficient alternate load path which effectively avoided progressive collapse. Furthermore, the concrete slabs were shown to have significant capacity even if they were vertically supported by three sides with the other unsupported side suffering from column loss. The crack patterns of three-edge supported slabs were studied, which can be also useful for further studies on the behaviour of the slabs supported along three edges.

Fig. 2.1 Illustration of tested composite frame, Astaneh-Asl et al. (2002)
A simple ‘robustness index’ that has a potential to be widely used by engineers was proposed to quantitatively evaluate the robustness of steel buildings under unknown extreme loadings (Iding, 2005). Three major weighted assessing criteria were introduced: redundancy, fireproofing and connection ductility, and each of these weighted aspects are scaled from ranking 0 to ranking 10, indicating the vulnerability from ‘very vulnerable to progressive collapse’ to ‘highest resistance against progressive collapse’. The advantage of this approach is its convenience for prescriptive guidelines that simply consider the type of structure without the complexity of performing structural analysis at least for preliminary evaluation.

Liu et al. (2005) conducted nonlinear dynamic analysis on typical multi-storey steel frames. It was suggested that the tying force requirement against progressive collapse recommended in the UK codes and building regulations (BSI, 2001; ODPM, 2004) is not sufficient. Although the study warns researchers and engineers of the deficiency of the tying force requirement, the rational of this finding needs further justification, since the benefits of the concrete slab and failure mechanism of joints were not considered in detail in the model. Employing a similar method, Kim and Kim (2009a, 2009b) assessed the potential for progressive collapse of gravity load resisting systems (gravity load is resisted by steel moment resisting frames while lateral load is resisted by shear walls) and lateral load resisting systems (the steel moment frames are designed to resist both gravity and lateral loads), where both of the systems are commonly employed against seismic loading. It was observed that the robustness of the structures designed for high seismicity is better than those for mid seismicity. A similar conclusion was also drawn by Khandelwal et al. (2008). This sheds light on the potential to connect seismic design to progressive collapse design in future.

Wang and Wald (2008) recognised deficiencies of the current robustness design method under fire conditions and suggested possible solutions. Two design philosophies were addressed, the first one was the control of progressive collapse from a structural design perspective, and the second one was the control of fire spread through fire isolation. Based on the first design philosophy, six feasible fire loading types were suggested for engineers, where a two-bay multi-storey framed structure was employed to illustrate the fire accidental loading, as shown in Figure 2.2. Among the six fire load scenarios, type $e$ can be the most typical one related to the robustness design of the structure under
localised fire. Types a and b are more associated with conventional fire engineering where single span beams under fire are considered and columns are not affected. Types c and d are simplified scenarios that only consider the column degradation under localised fire, and these may be used for preliminary robustness assessments. Finally, type f fire scenario is proposed, but this is fully developed fire which is beyond the scope of this study. Besides the proposed fire scenarios, significant design considerations were also highlighted in Wang and Wald’s research: 1) beam-column joints need to be designed with sufficient strength and ductility to ensure the formation of catenary action – if a column fails, rigidity of joints in conjunction with the resistance of the slab is critical for sufficient robustness; 2) due to the degradation of the fire affected beams, the column that is connected to the failed beams may have a buckling length of up to the height of two floors; and 3) if fire influences most of the structural members in a certain floor ('case f' in Figure 2.2), there will be a great potential for collapse, unless extra precautions are incorporated. With respect to the concept of preventing massive fire spread, it was suggested that fire isolations need to be sufficient, and failure of key components that isolate fire should be limited. This aspect is not considered in this study where car parks are selected as the reference building.

![Fig. 2.2 Possible fire scenarios for robustness design, Wang and Wald (2008)](image)

Sadek et al. (2008) discussed the contributions from different structural components (i.e. deck and concrete slabs) and confirmed the importance of 2D slab action. Three levels of numerical models were established, namely, framing only, framing with steel deck and framing with detailed floor model. Through successively incorporating the
contribution of the steel deck and the concrete slab, the influence of these components on robustness was studied. It was concluded that the composite floor plays two significant roles for enhancing the performance of the considered structures: 1) large inward displacements of exterior columns are effectively decreased via the diaphragm effect of floor slabs; and 2) the capacity of the floor system is significantly enhanced due to membrane action.

With the consideration of the effect of joint ductility, Foley et al. (2008) investigated the response of steel framed structures under interior column loss scenarios. It was noted that the smooth transition of joints from flexural bending to tensile action is important to ensure sufficient ductility supply, whereas a snap-through response of low ductility joints can induce a premature failure prior to the formation of catenary action. It was also assumed that the capacity of any floor system can be seen as a simple superposition of the slab and the steel skeleton under extreme conditions like fire. However, this assertion ignores composite behaviour, which can be significant even when the double-span floor system deflects significantly after column loss.

Izzuddin (2009) comprehensively reviewed the current specialist design codes and recent developments in performance-based assessments for structural robustness. It was noted that ductility is insufficiently considered in the traditional prescriptive methods (e.g. the ‘tie force approach’), which renders these conventional design methods potentially unsafe. It was also highlighted that although column removal scenarios in the newly developed codes (GSA, 2003; and DoD, 2009) are more performance-based and event-independent, they are computationally expensive and too demanding for design-oriented practice when addressed using nonlinear dynamic analysis. On the other hand, the simplified consideration of dynamic effects using dynamic increase factors (DIF) can sometimes be over conservative for some types of structures, or unsafe for others (Izzuddin and Nethercot, 2009). To address this, a more rational ‘Ductility-Centred’ Approach with a practical and multi-level assessment framework was proposed (Izzuddin et al., 2007, 2008; and Vlassis et al., 2008). This new method utilises three stages: 1) nonlinear static response, which considers the nonlinear response of the damaged structure under gravity loading; 2) simplified dynamic assessment, which transforms the obtained nonlinear static response to the maximum dynamic response, so-called pseudo-static response, via the principle of energy balance;
and 3) ductility assessment, which compares the joint ductility demands with the joint ductility supply at the maximum dynamic response. Therefore, only the nonlinear static analysis is required in this method, while the dynamic response can be obtained using the simplified dynamic assessment approach, which overcomes the shortcomings of both detailed nonlinear dynamic analysis and simplified dynamic increase factors as specified in conventional design codes. Furthermore, various levels of structural idealisations were proposed in this assessment framework, depending on design requirements and the feasibility of model reductions, as illustrated in Figure 2.3. In the context of robustness under localised fire, this approach (Izzuddin et al., 2008) forms a sound basis, but requires enhancement to account for elevated temperature in the fire affected floor as well as for the dynamic effects arising from column buckling under fire, which are likely to be different to the dynamic effects due to sudden column loss.

Fig. 2.3 Multi level models for structural progressive collapse assessment (Izzuddin et al. 2008)

Further to the work undertaken at Imperial College London by Izzuddin et al. (2008), Gudmundsson and Izzuddin (2010) demonstrated that the assumption of sudden column loss for progressive collapse analysis is conservative for scenarios where column damage is caused by blast. Through extensive parametric studies on 2D frame models, it was shown that sudden column loss offers an upper bound on the ductility demand in the upper floors. Towards a more rational and practical assessment of the dynamic
ductility demand, a correlation factor $\beta_r$, which was defined as the ratio of the maximum vertical displacement from a real dynamic blast loading scenario over that from a sudden column loss scenario, was proposed. Making use of the multi-level concept (Izzuddin et al., 2007, 2008; Vlassis, 2007), Luu (2008) developed a three-level 2D modelling system (illustrated in Figure 2.4) with different modelling idealisations. Analysis was performed to evaluate the boundary conditions which represent the restraints provided by the unaffected parts of the structure, in order to ensure that the models of lower levels (levels 2 and 3 in Figure 2.4) behave as close as possible to the higher level model. Two parameters, the lateral restraint stiffness $K$ and the corresponding resistance $F_{Rd}$ of the axial restraints, were proposed, and a quantitative calculation approach was developed to evaluate the two parameters in the 2D steel framework. However, unlike the multi-level approach developed by Izzuddin et al., (2007, 2008), the proposed method does not consider the contribution from the in-plane diaphragm effect of the neighbouring floor system as well as the axial stiffness provided by the adjacent joints.

![Fig. 2.4 Three-level 2D modelling system (Luu, 2008)](image)

<table>
<thead>
<tr>
<th>Level 1</th>
<th>Level 2</th>
<th>Level 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame’s response in Load Phase 3</td>
<td>Directly affected part substructure</td>
<td>Isolated membrane beam level</td>
</tr>
</tbody>
</table>

Employing the multi-level robustness assessment framework proposed by Izzuddin et al. (2008), Yu et al. (2010) investigated the robustness behaviour of a typical steel-composite moment resisting framed building subject to sudden column loss, where emphasis was given to the influence of three different structural modelling levels, namely, beam assembled model, grillage model, and full slab model. It was observed that the beam assembled model behaved similarly to the grillage mode, but both of them underestimated the progressive collapse resistance of the building due to the disregard
of 2D slab membrane action. On the other hand, the full slab model could predict a more reliable structural performance, but it is more computationally expensive. In order to evaluate the influence of the 2D slab action, parametric studies were performed, and a resistance increasing factor was considered which is expressed by the ratio of the gravity load resistance of the 2D slab over that of the grillage slab. Based on this study, a typical resistance increasing factor of around 2.0 was observed for the considered structure. Furthermore, the effect of edge boundary conditions on slab resistance was evaluated, where the ‘reduced slab models’, consisting of the directly affected bays only, with various boundary conditions were compared with the full structural models comprising both the affected floor slab and the neighbouring slabs. It was found that the progressive collapse resistance of the reduced slab model with both rotational and planer restraints applied along the slab edge can overestimate the actual response, whereas a reasonable yet conservative prediction can be achieved when only planar restraints were considered for the reduced slab model.

Yu et al. (2009c) investigated the influence of joints and composite floor slabs on the progressive collapse resistance of a 3D composite structural model, as shown in Figure 2.5. For the joints, the effects of pin, semi-rigid and rigid joints on the effective tying force developed in the steel beams were studied. Regarding the floor slabs, shell elements were employed, and the influences of different concrete material properties, steel deck profiles, and reinforcement ratios were investigated. It was suggested that progressive collapse resistance can be improved by introducing more rigid connections. This conclusion is, however, questionable since joint ductility was not comprehensively considered in the model. The results also showed that the decking profile with higher flexural bending moment resistance has better progressive failure resistance, and locally increasing the reinforcement ratio in the vicinity of joints can enhance the structural robustness effectively. Finally, it was observed that employing concrete with higher tensile strength is beneficial, while the compressive strength of concrete has much less influence.
Liu (2010a) reaffirmed the widely held belief that the formation of the catenary action is significantly beneficial for structures against progressive collapse, but for most existing steel/composite buildings in the UK, enhancing structural robustness through improving the structural behaviour during the catenary action stage can be difficult due to the weakness of the widely adopted simple beam-to-column joints. These joints may have poor ductility capacity before the forming of catenary action, which leads to a premature joint failure followed by progressive collapse. To address this, two retrofitting schemes were proposed to strengthen simple connections in existing buildings, as shown in Figure 2.6. The aim of both schemes was to transform nominally pinned joints to full-strength joints. Finite element substructure models were utilised (Liu, 2010b) in order to further verify the effectiveness of the proposed approach. Based on these numerical studies, it was suggested that catenary action can be fully formed after introducing the retrofitting schemes. Galal and EI-Sawy (2010) also proposed several retrofitting strategies, but the solution was to strengthen the beams instead of joints in existing structures. Two methods were found to be useful, namely, the ‘Fibre-Reinforced Polymer (FRP) composite method’ and the ‘additional continuous steel plate method’. However, the conclusion was only based on the load-deflection response without considering the joint ductility, thus the effectiveness of the two methods needs further investigation.

**Fig. 2.5** Illustration of FE model subject to column loss (Yu et al. 2009c)
Izzuddin et al. (2000) and Song et al. (2000) discussed the fire resistance of steel structures subsequent to blast. Through investigating a 2D steel-framed structure, it was observed that the damage induced by moderate explosion loading can significantly influence the subsequent fire resistance of the affected steel columns. Inspired by the work of Izzuddin et al. (2000) and Song et al. (2000), Liew (2008) further investigated the survivability of steel-framed buildings subject to blast and subsequent fire. A mixed-element modelling strategy was proposed for the overall steel-framed model, where shell elements were used in the directly affected part, and beam elements were adopted for the unaffected part. Through the investigation of the mixed-element model, the post-blast fire response of the damaged column was considered, where it was found that negligible fire resistance is retained in a column that is significantly damaged by blast. Similar research was also undertaken by Quiel and Marjanishvili (2010), who investigated the response of structures subject to fire hazards followed by a severe localised damage on columns. Instead of carrying out a dynamic analysis on columns subject to blast load, it was conservatively assumed that the targeted column has already been fully damaged and could be removed completely; afterwards the fire loading was applied on the double-span system. Ambient analysis was conducted first, and it was observed that the considered model had sufficient robustness under pure ‘sudden column loss’ scenarios without the consideration of subsequent fires. Afterwards, fire loading was applied, and several protection schemes were proposed. It was indicated that even if a structure has sufficient resistance against progressive collapse under ambient column loss scenarios, subsequent fire hazards could lead to total collapse of the structure if adjacent members are exposed fire without protection. However, this conclusion was only based on simplified 2D models without considering the 2D slab
membrane action as well as the joint details. As discussed in Chapter 7, this study may give misleading results under specific circumstances.

Fig. 2.7 Demonstration of column loss test (Jaspart et al., 2008)

With the goal of evaluating the role of joint ductility in mitigating the potential for progressive collapse of composite buildings subject to extreme loading, and thus to provide practical requirements and design recommendations from a robustness point of view, a European RFCS project named ‘Robust Structures by Joint Ductility’ was launched (Kuhlmann et al., 2009). This project involved a series of systematic research including experimental, numerical and analytical investigations. Firstly, tests on composite substructures reproducing the scenario of an internal column removal were conducted (Jaspart et al., 2008), as shown in Figure 2.7. A double-span subsystem was established, where the adjacent unaffected structures were replicated by two axial restraints of which the stiffness was provided by horizontal jacks. End-plate joints were employed for beam-column connections. With the application of the vertical point load, four distinct stages were observed, namely, elastic stage, yielding stage, plateau stage, and catenary stage. At the first stage, small deflections were developed, and slight concrete cracking was observed in the vicinity of the hogging joints, but the whole subsystem remained largely within the elastic range. Subsequently, yielding of steel components (e.g. column web in compression) was initiated. This was followed by a plateau range, where concrete cracking was further developed. Finally, the subsystem stepped into the catenary stage until the reinforcement rebars were completely ruptured. A certain level of resistance was retained when the steel connections worked alone, and
the ultimate rotation of the beam was approximately 190mrad. The test result reflected sound joint ductility, but it is noteworthy that the tested steel joints were reduced in size, so the actual ductility of the joints with normal sizes may be different. In parallel with the tests on the subsystem frame, experimental investigations on individual joints were also carried out (Kuhlmann et al., 2007). Along with numerical verifications, it was found that the post-yielding resistance of ductile components were considerable, so it was recommended that ductile failure modes should govern the ultimate failure of joints to ensure greater ductility supply.

Alashker et al. (2010) established a detailed finite element model (as shown in Figure 2.8) with the aim of assessing the progressive collapse resistance of steel-composite frameworks via parametric studies. The accuracy of the composite slab modelling technique was validated first through comparison with several related tests, and sound correlations were observed. Subsequently, the influences of several parameters were investigated, including the steel deck thickness, the slab reinforcement mesh, and the number of bolts in beam-to-column fin-plate connections. It was found that increasing the thickness of the steel deck and reinforcement mesh could mitigate the potential for progressive collapse, but only when there is sufficient capacity and ductility of the connections can the increased resistance of the 2D slab be fully mobilised. Finally, it was found that the benefit from only increasing the number of bolts in fin-plate connections was rather limited.

![Finite-element quarter-model of floor system (Alashker et al. 2010)](image)
2.3 Fire design and assessment

2.3.1 Current fire design procedures

To ensure safe and economic design of structures under fire conditions, design approaches with different levels of sophistication have been developed and included in current design guidelines (BSI, 2003; Eurocode 1, 2002; Eurocode 3, 2005b; Eurocode 4, 2005). Two main approaches are usually employed, namely, the prescriptive approach and the performance-based approach. At present, the prescriptive approach is still widely used, but it is only applicable for isolated members. On the other hand, the performance-based approach offers a more popular solution for modern fire design. Generally, three stages are involved in the current performance-based fire design framework:

*Fire modelling.* Fire scenarios should first be proposed to determine the heat release conditions and the resulting air temperatures. The simplest way is to use nominal fire curves, which does not require any physical parameters. This method assumes a uniform temperature distribution in members, and subsequent thermal analysis is not required. With appropriate modifications, this method can form the basis of the simplified Temperature-Dependent Approach for localised fire conditions, as discussed in Chapter 6. A more sophisticated method is the time-equivalence approach, which attempts to reflect the effect of real fire based on the standard fire curve through considering several important parameters such as fire severity, ventilation conditions, thermal properties of boundaries, and compartment sizes. An even more sophisticated fire modelling approach, namely the ‘compartment fire model’ approach, is also applicable to simple calculations using for example a spreadsheet. This approach is capable of considering some additional parameters (e.g. ceiling height) for localized fire scenarios, and is thus used in the detailed Temperature-Dependent Approach as discussed in Chapter 5. More rational approaches include zone models and computational fluid dynamics (CFD) modelling, though these are often too computationally demanding for practical and regular application in structural fire design.

*Thermal analysis.* Thermal analysis is undertaken as a subsequent stage to predict the temperature profile within the structural members (e.g. steel beams, steel columns, steel decks and concrete slabs). Three approaches with increasing complexity are commonly employed, namely, design charts, simple formulae, and advanced models. The simplest
method of obtaining the temperature distribution in structural members is to use the test data provided in tables or charts which are published in codes, design guidance, and research reports. Such test data are mainly based on standard fire conditions. The parameters required for the design chart method normally include construction types and member geometries, so fire modelling is unnecessary in this method. The simple formulae approach is to predict the temperature profile in structural members through simple design equations specified in design codes (Eurocode 1, 2002). This approach is more accurate compared to the design chart approach, because it additionally considers the parameters of fire curves, boundary conditions, and material thermal properties. The most advanced and reliable approach is based on heat transfer finite element analysis, as is used in the detailed Temperature-Dependent Approach in Chapter 5. This yields the most accurate solution, but can be computationally demanding and requires relevant expertise to ensure that the models are applied correctly.

*Structural analysis.* The final stage of fire assessment is determining the structural response under the resulting temperature distribution. Three modelling approaches, with increasing complexity, are commonly utilised, namely, simple element models, sub-structure models, and advanced finite element models. The simple element approach investigates individual members at the fire limit state. Input parameters for an individual member (i.e. steel/composite beams, steel/composite columns and composite slabs) include the applied mechanical loading, temperature profiles over the member cross-section, material degradation properties, and simplified boundary conditions. With these input parameters, simple calculations can be performed for the ultimate strength. This approach is not able to precisely reflect the interaction between structural members but is considered to be straightforward and conservative; therefore, it is widely accepted in design practice. Instead of using individual member models, a sub-structure model can be established provided that temperature distributions within the sub-structure and boundary conditions are carefully modelled. This approach considers the actual load path and restraints, and is applicable to simplified hand calculations or simple computer-aided models. Finally, the most accurate approach for predicting the behaviour of buildings under fire conditions is based on advanced finite element models, which are used in this study for structural analysis. This approach incorporates full material stress-strain-temperature relationships, thermal expansion properties, and detailed temperature profiles, and it can predict the strains, stresses and deformations for
all members over the duration of fire. However, localised behaviour (e.g. reinforcement ruptures and joint failures) require careful attention, as it may not be incorporated in general finite element (FE) models.

### 2.3.2 Localised fire

A building fire is likely to remain localised provided that the compartment or opening is sufficiently large. A localised fire has its unique thermal characteristic which is different from the flashover compartment fire. Eurocode 1 (2002) provides a simple method for determining the thermal action of localised fires in Annex C (informative). A localised fire can be defined as either a small flame where the flame height is less than the ceiling height, or a large flame where the fire impinges on the ceiling of the compartment, as shown in Figure 2.9. For a small flame, the design method based on the work of Heskestad (1995) can be utilised to calculate the temperatures in the plume at various heights along the vertical flame axis. For the large flame, Eurocode 1 (2002) only provides design formulae to determine the heat flux received by the surface area at the ceiling level, based on research of Hasemi et al. (1995).

![Illustration of localised fires](image)

**Fig. 2.9** Illustration of localised fires (Eurocode 1, 2002). a) Localised fire not impacting the ceiling; b) Localised fire impacting the ceiling.

More emphasis is given in this work to the second case where localised fire may impact the surfacing of ceilings, because this can cause more damage to the floor systems. The obstruction of the ceiling surface forces the heat flow to move horizontally from the flame axis to remote areas of the ceiling. The gas flow in a shallow layer beneath the ceiling surface is called the ‘ceiling jet’ (Evans, 1995). One of the interests of localised fire modelling is to obtain the ceiling jet temperature or heat flux distributions along the
radius from the flame axis. Generally speaking, the ceiling jet temperature and heat flux tend to decrease nonlinearly from the maximum temperature near the axis of the fire origin towards the outer cooler perimeter of the ceiling area. So far, the ceiling jet temperature decreasing rate along the radius of the ceiling above a one-origin localised flame has been intensively investigated.

Towards the goal of optimizing the ceiling-mounted fire and smoke detectors, Alpert (1972) developed empirical equations for determining the temperature distributions in the ceiling jet flow produced by localised fire. The study is based on extensive experimental data, and the proposed ceiling jet temperature distribution is given by:

\[
\Delta T = \frac{16.9Q^{2/3}}{H^{5/3}} \quad \text{for } r/H \leq 0.18 \quad (2.1)
\]

\[
\Delta T = \frac{5.38(Q/r)^{2/3}}{H} \quad \text{for } r/H > 0.18 \quad (2.2)
\]

Heskestad and Delichatsios (1978) developed a different empirical equation which appears to be the most reliable one based on the number and scale of experiments performed (Beyler, 1986), where:

\[
\Delta T = \frac{2.75Q^{2/3}(0.188 + 0.313r/H)^{-4/3}}{H^{5/3}} \quad (2.3)
\]

Cooper (1982) also estimated the unconfined ceiling jet temperature:

\[
\Delta T = 28.1Q^{2/3}H^{-5/3}e^{-1.77r/H} \quad \text{for } r/H \leq 0.75 \quad (2.4)
\]

\[
\Delta T = 5.77Q^{2/3}H^{-5/3}(r/H)^{-0.88} \quad \text{for } r/H > 0.75 \quad (2.5)
\]

For all the above equations, \(Q\) is the rate of the heat release in a fire, \(H\) is the ceiling height, \(r\) is the radial location from the flame axis, and \(\Delta T\) is the ceiling jet temperature increase.

For comparison purposes, the temperature-distance relationships from the above three research results are summarised in Figure 2.10, where the value of \(T_r/T_{max}\) represents the ratio of the jet flow temperature at the location of radius \(r\) over the maximum
temperature near the flame axis, and \( r/H \) stands for the ratio of the distance from the flame axis over the ceiling height.

![Fig 2.10 Ceiling jet temperature distributions at distances from the flame axis](image)

Wakamatsu et al. (1996) undertook a series of experiments to measure the temperature distributions of steel beams (underneath floor ceilings) exposed to localised fires. Particular interest was given to the variation of the heat flux and surface temperature along the beam length. The experimental setup is demonstrated in Figure 2.11. The diameter of the burner was approximately 0.5 m, and different heat rates (95kW, 100kW, 130kW, 150kW, 160kW, and 200kW) and distances (1m and 0.6m) between the burner surface and the bottom of the beam were tested. Selected results are summarised in Figures 2.12 and 2.13 showing the heat flux vs. distance and the bottom flange temperature vs. distance distributions, respectively.

![Fig. 2.11 Experimental setup of floor exposed to fire (Wakamatsu et al., 1996)](image)
Sokol et al. (2008) performed a localised fire test on an internal column in an industrial building which was ready for demolition, as shown in Figure 2.14. The test was conducted to obtain the temperature profile within the steel column as well as the temperature distribution of the gas along the adjacent beams. The column was unloaded for temperature measurements only. It was observed that the maximum temperature in the column reached 500°C. The maximum gas temperature underneath the steel beams was between 700°C and 800°C, this gas temperature decreased by 300°C at a distance of 1.5m from the flame axis, and by 400°C at the distance of 2.5m. The test was followed by parametric studies using the numerical tool ANSYS, and the conclusions
include: 1) the temperature distribution along the column length can be seen as uniform provided that the length of the column is similar to the length of the fire flame, 2) a thermal gradient over the column cross-section is caused during the localised fire when the location of the fire origin is above a specific distance away from the column.

![Localised fire test on a column (Sokol et al. 2008)](image)

**Fig 2.14** Localised fire test on a column (Sokol et al. 2008)

A common finding from these investigations on localised fire is that the area of high temperature beneath the floor ceiling is normally confined within a small zone immediately above the fire source. Other structural parts beyond this localised area are much cooler because the temperature can decrease rapidly towards the outer regions. This finding has been helpful towards the definition of a simplified temperature distribution, as presented in Chapter 6.

### 2.3.3 Structural member response under fire

#### 2.3.3.1 Columns

Alternative multilevel modelling approaches can be employed to check the capacity of columns under fire conditions, specifically simple element models, sub-structural models, and advanced global finite element models. Comparing to the latter two sophisticated assessment approaches, the simple element method is more widely employed in design-oriented application due to its simplicity and efficiency. The structural design procedures (e.g. in Eurocode 3, 2005b) with a simple column model is mainly comprised of the following two simple steps.
1) Predict the maximum temperature within the column based on the fire resistance class.

2) Check if the design load is less than the design column resistance under fire

\[ N_{fi,d} \leq N_{b,fi,Rd} \]

where \( N_{b,fi,Rd} = \chi_{fi} A_n k_{y,\theta,\text{max}} f_y / \gamma_{M,fi} \), in which \( \chi_{fi} \) is the reduction factor for flexural buckling, and the effective length depends on the location of the fire floor in a building, \( A_n \) is the cross-sectional area, \( k_{y,\theta,\text{max}} \) is the reduction factor for the yield strength of steel at the maximum steel temperature, \( f_y \) is the ambient yield strength, and \( \gamma_{M,fi} \) is the material factor.

This assessment approach is based on the capacities of isolated heated columns, while the conditions of end restraints, especially axial restraints, are not considered. This may lead to a pronounced inaccuracy in predicting the fire response of a thermally expanded column with axial restraints induced by the connected beams and slabs. The axial restraints can greatly influence the failure temperature of a column due to the additional compressive force generated through thermal expansion. Moreover, \( P-\Delta \) effects can be amplified by initial crookedness and imperfection due to thermal expansion. On the other hand, the axial restraint may conversely retard the failure of a heated column by the ‘pulling up’ effect in the post-buckling stage; this phenomenon has begun to attract more attention particularly for the concern of structural robustness. In view of the above reasons, current design codes suggest that advanced calculation models can be employed for fire resistance check on steel columns, accounting for member interactions between the columns and the connected members. Intensive research has been undertaken on this front.

Neves (1995) undertook a series of numerical analyses to investigate the fire response of columns with different axial restraints, column slenderness and restraint eccentricities. It was proposed that the fire resistance time of a column could be defined at the point when the compressive force falls back to the initial value under ambient condition. Based on this assumption, it was concluded that the column fire resistance decreases with the increasing of the axial restraint stiffness and column slenderness. Similar conclusions were also drawn by Ali et al. (1998), Tan et al. (2007), Huang and Tan (2007), Korzen et al. (2009), and Carvalho et al. (2009). However, a new finding was presented by Franssen (2000) with respect to the influence of axial restraints. It was
observed that the restraint stiffness has limited effect on the prediction of the failure temperature, and this was verified by intensive numerical models with different axial restraints as shown in Figure 2.15, where $R$ represent the ratio of axial restraint stiffness over the column axial stiffness. Since it was assumed that the column failure occurs when the axial force reduces back to the initial ambient value, it was found that the curves intersect at a similar failure point. Therefore, it was concluded that the column failure temperature is not sensitive to axial restraints.

![Failure temperature for different restraint degrees](image)

**Fig. 2.15** Load-temperature curves for columns with different axial restraints (Franssen, 2000)

However, the abovementioned previous research did not consider the post-buckling behaviour of the columns. To address this, Wang (2004) discussed three stages observed in axially restrained columns under fire, namely, pre-buckling stage, buckling stage and post-bucking stage. The critical temperature was also assumed to be at the point where the column axial force reduces back to the initial ambient value. Through parametric studies, it was indicated that for most practical situations, where restraint stiffness is usually less than 3% of the stiffness of the column and the load ratio is larger than 0.5, the defined critical temperature is almost equal to the buckling temperature, while limited further resistance after column buckling is expected. A different conclusion was drawn by Huang and Tan (2004) and Huang *et al.* (2006) who defined the critical temperature in a similar way as given in Figure 2.16. It was found that heated columns may remain stable even if the critical temperature is exceeded, provided that the end connections remain intact. The diverse conclusions from the two
researchers may be attributed to the different assumptions of the column end rotational restraints, load ratios, and slenderness ratios.

![Diagram](image)

**Fig. 2.16** Development of column internal axial force (Huang and Tan, 2004)

Li *et al.* (2010) conducted two fire tests with the aim of investigating the buckling and post-buckling response of steel columns subject to different axial restraint stiffness. As expected, the buckling temperature of the column with lower axial restraint stiffness was relatively higher due to a smaller generated compressive force. However, benefit was observed in the column with greater restraint stiffness after buckling, where a new equilibrium state was more quickly achieved. After new equilibrium was reached, a slower rate of lateral deflection increase was further developed in the column, and the ultimate failure temperature of the column was effectively postponed. Further to fire tests, extensive parametric studies were undertaken by Wang *et al.* (2010), where the axial load ratio and the column slenderness were additionally investigated. It was concluded that the gaps between the ‘buckling temperature’ and the ‘failure temperature’ (where nearly all the resistance is lost) of axially restrained columns are more considerable for those with high axial restraint stiffness, high slenderness, and small load ratios.

### 2.3.3.2 Floor systems

In the current design codes such as Eurocode 4 (2005), the floor system is usually assessed in terms of isolated members (i.e. isolated composite beams or isolated unidirectional composite slabs) analysed by simple calculation methods. The typical fire assessment procedure for a composite beam is as follows.
1) *Use load combinations to obtain the mechanical load applied on the composite beam during fire exposure.*

2) *Calculate the temperature distributions across the cross-section.* Usually the temperature profile within the concrete slab is nonlinear, while the temperature over the cross-section of the steel beam can be taken as uniform. Due to the nonlinearity of the concrete slab temperature field, the depth of slab can be divided into several layers, and each layer has a uniform temperature.

3) *Fire resistance checking.* The strength of shear connecter and the bending capacity of the composite beam are checked. The procedures are similar to those under ambient temperature, but the degradations of strength and stiffness need to be considered. Thermal expansion effects are ignored in this simplified assessment method, because isolated composite beams are usually considered as simply-supported.

The above design procedure of composite beams should be followed by the checking of the slab spanning perpendicularly between the supporting beams. For a typical orthotropic composite floor system with steel deck, 2D effects (e.g. the membrane action) are usually ignored in the current codes for conservative purposes, so the slab supported between the beams can be seen as unidirectional. Therefore, the design process for the slab is similar to that dealing with one-dimensional beams. A commonly employed design process for an orthotropic composite slab is summarised as follows.

1) *Use load combinations to obtain the mechanical load applied on the composite slab during fire exposure.*

2) *Determine the temperatures in concrete, reinforcement, as well as the upper flange, web, lower flange of steel deck* (see Figure 2.17).

3) *Fire resistance checking with respect to ‘the thermal insulation criterion’.* This criterion is to satisfy the thermal insulation condition of the member. In the context of this criterion, the average temperature of 140°C within the slab and the maximum temperature of 180°C at the top of slabs should not be exceeded.

4) *Fire resistance checking with respect to ‘the mechanical criterion’.* In the context of this criterion, both the hogging and sagging moment resistances need to be calculated.
In some cases, only a reduced cross section of the slab is considered, while the steel decking and the concrete part above a certain temperature are removed.

![Illustration of composite slab under checking in Eurocode 4 (2005)](image)

**Fig. 2.17** Illustration of composite slab under checking in Eurocode 4 (2005)

The above design approach can be too conservative, especially for fire and progressive collapse, because several important aspects are ignored (e.g. the membrane action). To address this, extensive research has been conducted over the past two decades.

Bailey (2001) developed a new design method for predicting the load carrying capacity for simply-supported slabs without planar restraints. It was assumed that a full depth crack is located at the centre location parallel to the shorter span, and ‘rigid slab parts’ are isolated by the yield lines and the central full depth crack. Through solving the equilibrium equations between these rigid parts, the maximum load capacity of the slab can be obtained using energy principles. Based on this work, an extended design approach dealing with the membrane action of composite floor systems under fire was further developed by Bailey (2004). Four behavioural stages of the floor system during fire were proposed, as illustrated in Figure 2.18. Under ambient conditions, the composite slab spans between the beams in a one-way manner. As the temperature begins to increase, a plastic hinge forms at the centre of the composite beam, and a fan yield pattern appears in the slab. Then, the moment resistance of the plastic hinge reduces as the temperature keeps rising, and a ‘cross’ yield line pattern forms. As the temperature increases further, the stiffness of the steel beam is completely lost, and the yield pattern of the slab tends towards the ‘back-of-envelopes’ shape which is similar to the case without the supporting beam. The load carrying capacity of the floor system $\omega_{p0}$ can be calculated from the energy equation as:

$$
\omega_{p0} = e_{sys} \left( \frac{W_s + W_k}{W_e} \right)
$$

(2.6)

where $e_{sys}$ is an enhancement factor considering slab membrane action, $W_s$ is the internal work done by the composite slab in bending, $W_k$ is the internal work done by
the beams in bending, and \( W_e \) is the external work done per unit load. The new design method was proved to correlate well with the selected experiment (the BRE corner fire test in Cardington), but simple computer-aided tools may be required. However, this method lacks a comprehensive failure criterion. It simply utilises an empirical approach for failure assessment, where a maximum value was defined for the average strain in the reinforcement without considering the strain concentration at crack locations.

\[
\text{Fig. 2.18 Behaviour of slab/beam floor system in fire, Bailey (2004)}
\]

Usmani and Cameron (2004) developed a new three-step theoretical method estimating the capacities of reinforcement slabs subject to fire. The three steps were: thermal input, mechanical response and load capacity estimation, and the slab capacity can be given by:

\[
q_{slb} = \frac{\Pi_{int}}{4LB} \frac{w_q}{\pi^2}
\]

(2.7)

Where \( \Pi_{int} \) is the total internal work \( \Pi_{int} = \sum_{n=1}^{\text{no.rebar}} \int_{\text{V}} \Delta \sigma \Delta \varepsilon \, dV \), \( w_q \) is the deflection caused by the design load, \( L \) and \( B \) are the length and width of slab, and \( \Delta \sigma \) and \( \Delta \varepsilon \) are the stress and strain excluding those induced by thermal effect (i.e. total stress and strain - thermal stress and strain). The energy principle was employed for determining the capacity. The tensile strength in concrete was ignored and only reinforcement bars were assumed to resist the overall load, which they considered to be reasonable under
CHAPTER 2 Literature Review

extreme load conditions such as fire. However, the bond conditions between the concrete and reinforcements as well as the stress concentration caused by concrete cracking were not considered in this approach.

Izzuddin et al. (2004) developed a new shell element which was implemented in the nonlinear finite element program ADAPTIC (Izzuddin, 1991) for simulating steel-decked composite floor slabs under extreme loading. The new element considers geometric orthotropy, compressive nonlinearity of concrete, concrete crack opening and closure, as well as elevated temperature effects. Additional rib freedoms in conjunction with the conventional freedoms typically associated with shell elements were incorporated into the new element to deal with geometric orthotropy, which was shown to offer sufficient accuracy and efficiency compared with 3D solid elements. The accuracy and reliability of the new element have been validated by comparisons against experimental results (Elghazouli and Izzuddin, 2004a). Subsequently, the new shell element was employed in composite floor system models to simulate the compartment Cardington fire test as well as isolated simply supported ribbed slabs, and good correlations were also found.

To further consider the bond characteristics and rupture behaviour of reinforcement, Izzuddin and Elghazouli (2004) developed two new analytical models, detailed and simplified, for nonlinear analysis of axially restrained lightly reinforced concrete (LRC) members under both ambient and elevated temperature conditions. Although consideration was mainly given to one-dimensional LRC members, this work was a significant step towards a more comprehensive understanding of the failure mechanism of two-dimensional composite slabs. For the considered LRC beam, the simplified model ignores the compressive arching stage, while the detailed analytical model considers both compressive arching and tensile catenary stages, and emphasis was particularly given to the failure assessment of reinforcement rupture. For both detailed and simplified analytical models, a single-crack mode was assumed. The two models were then verified against the numerical predictions of ADAPTIC. It was observed that the detailed model compares accurately with the ADAPTIC predictions in both the compressive arching and tensile catenary stages, provided that the reinforcement ratio is not too high. For members with higher reinforcement ratios, the analytical model is shown conservative. The simplified model, which is more design-oriented, was shown
to be capable of providing a close comparison against the ADAPTIC results in the tensile catenary stage. So the simplified model can be employed for practical assessment and design with sufficient accuracy if reinforcement rupture does not occur within the compressive arching stage. The study was then followed by a series of parametric investigations with the aim of highlighting several key parameters that affect the member response under both ambient and elevated conditions (Elghazouli and Izzuddin, 2004b). It was concluded that failure of concrete members can be mitigated in the presence of lower bonding force or greater steel strain hardening. In addition, the reinforcement ratio, member depth and length, thermal expansion effect, and material degradation property were also shown to have significant influences on the failure behaviour of concrete members subject to fire.

Huang et al. (2004) established several floor system models using the finite element program VULCAN developed in the University of Sheffield. Different secondary beam protection regimes, reinforcement meshes, and localised fire locations were considered. It was found that the level of reinforcement mesh has limited influence on the slab resistance below the temperature of about 500°C to 650°C. However, as the temperature increases beyond, the level of reinforcement mesh becomes a key factor enabling tensile membrane action which can effectively reduce the deflection. It was also observed that whether membrane action can develop depends on aspect ratios (length over width ratios) of the slab. With high aspect ratios, catenary effects instead of 2D membrane actions are more likely to happen. Based on the results considering different fire locations, it was shown that the external-bay slab experiences a larger deflection than that of the internal bay slabs. This is due to the fact that less planar restraints from cooler areas are applied on the external slab.

Li et al. (2007) summarised possible developing stages of membrane action of the slabs with full planar restraints under fire (as shown in Figure 2.19). As temperature starts to increase, cracks begin to spread in the surface of concrete and yield lines are formed. Subsequently, compressive membrane effects start to be dominant due to thermal expansion of the slab resisted by the surrounding restraints along the edge. As the deflection increases, tensile membrane action replaces compressive membrane action, and similar to a tension net, the reinforcement mesh starts to resist the vertical loading. If planar boundary restraints are not applied, an outer compression ring around the slab
can balance the tensile membrane action in the middle. As the deflection further increases, the yield lines start to diminish with the spread of the cracks. At the last stage, due to the massive cracking of concrete as well as the decreasing strength of concrete subject to fire, nearly all the gravity loading is resisted by the reinforcement mesh within an elliptic zone. The magnitude of membrane action depends on the boundary conditions of the slabs, where the existence of planar restraints can encourage more considerable tensile membrane action. Considering common practice, composite slabs are connected with the surrounding beams via shear studs, and the internal slabs are normally continuously casted with the surrounding slabs. Therefore, it can be reasonable to assume that planar restraints are applied for internal slabs.

![Diagram of slab membrane response under fire](image)

**Fig. 2.19** Development of slab membrane response under fire (Li et al. 2007)

Omer *et al.* (2009; 2010a; 2010b) developed an analytical model for predicting the failure behaviour of 2D simply supported LRC slabs with and without planar edge restraints, which offered for the first time a rational approach to failure assessment of floor slabs under fire by fracture of reinforcement. This work is an extension of the previews work on one-dimensional LRC beams (Izzuddin and Elghazouli, 2004). The analytical slab model considers the influence of steel-concrete bonding, slab membrane action, and elevated temperature effects. In order to enable practical application, simplified models were also proposed as alternatives. Both the detailed and simplified analytical models compared well with the finite element model in ADAPTIC as well as a few available experiments. Through several parametric investigations, it was found that bond strength and reinforcement strain hardening are the two most important
factors that influence the resistance of LRC slabs, based on the failure criterion governed by the rupture of reinforcements.

2.3.4 Fire tests

![Fig. 2.20](image)

**Fig. 2.20** Full-scale tested structure employed in Cardington experiment: (a) structural layout, (b) test 1, (c) test 4 (BRE, 1994; BRE, 1996)

In recent fire accidents such as the Broadgate fire and the Churchill Plaza Building fire in the UK, the structures in the absence of fire protection successfully survived under high temperatures. This indicates that traditional fire design codes based on the behaviour of isolated members subject to fire are grossly conservative. In this context, some researchers indicated that due to member interactions, the structures subject to localised fire may not suffer from overall damage or progressive collapse even if several members are heavily damaged. In order to investigate the overall structural behaviour considering the interaction between the directly affected part and the adjacent sub-structures, the ‘Cardington experimental programme’ was launched (BRE, 1994; BRE,
1996; Martin and Moore, 1997) on a full-scale eight-storey steel framed composite building, as shown in Figure 2.20. The individual fire tests on the tested building are briefly described next.

Four tests were first performed. In test 1, the fire was located on the 7th floor and the tested sub-structure involved a restrained universal beam and the surrounding concrete floor with a 9m span between the columns. This test was aimed at studying the behaviour of individual composite beams subject to fire. Test 2 considered a plane frame consisting of a series of beams and columns on the fourth floor. During the test, a maximum temperature of 750°C was reached in the primary and secondary beams as well as unprotected connections. Test 3 involved a localised fire carried out in the south east corner of the frame on the first floor. A maximum temperature of 935°C was recorded in the beams, and the deflection recovered from 428mm to 296mm after the cooling phase. Test 4 considered an office fire scenario, where the unprotected beams reached a maximum temperature of 1150°C, and a maximum deflection of 640mm was observed. Apart from these four tests, two other tests, test 5 and test 6, were conducted subsequently. Test 5 was a corner fire test at floor level 3. The duration of the fire lasted approximately 114 minutes, and the maximum air temperature and member temperature were 1000°C and 903°C, respectively. The maximum deflection recorded was 269mm. Test 6 was a large area compartment fire experiment which involved an area of 340m² at the same level of test 5. The whole test lasted 70 minutes and the maximum temperature of steel reached 691°C. A maximum beam deflection of 557mm was recorded. Finally in January 2003, test 7 was initiated to investigate the integrity of the structure as well as the structural behaviour of beam-to-column joints at elevated temperatures. Although progressive collapse resistance was not a prior concern in most Cardington tests where columns were protected, the seven tests shed considerable light on actual member interaction response under fire, and provide valuable benchmarks for further numerical investigations. Apart from the Cardington tests, a variety of other fire tests with variable scales have been carried out with the aim of either evaluating the structural response under elevated temperature or investigating the temperature distribution within the structures. The scales of the structures being tested ranged from 2-D steel sub-frames to delicately constructed full-scale buildings. A summary of the fire tests is provided in Table 2.2.
<table>
<thead>
<tr>
<th>Fire tests (year)</th>
<th>Country</th>
<th>Type of fire</th>
<th>Type of tested structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Butcher et al. (1968)</td>
<td>UK</td>
<td>Localized fire triggered by car</td>
<td>Steel scaffolding structure</td>
</tr>
<tr>
<td>Nippon Steel,1970 (ECCS,1993)</td>
<td>Japan</td>
<td>Localized fire triggered by car</td>
<td>steel framed parking building</td>
</tr>
<tr>
<td>Gewain (1973)</td>
<td>USA</td>
<td>Localized fire triggered by car</td>
<td>multi-storey composite car park</td>
</tr>
<tr>
<td>Genes (1982)</td>
<td>USA</td>
<td>Furnace compartment fire</td>
<td>two-storey compartment replicating a 20-storey building</td>
</tr>
<tr>
<td>Koike et al. (1982)</td>
<td>Japan</td>
<td>Electrical furnace fire on specific members</td>
<td>2D steel frame (substructure)</td>
</tr>
<tr>
<td>Ooyanagi et al. (1983)</td>
<td>Japan</td>
<td>Electrical furnace fire on specific members</td>
<td>3D steel frame (substructure)</td>
</tr>
<tr>
<td>Hirota et al. (1984)</td>
<td>Japan</td>
<td>Electrical furnace fire on specific members</td>
<td>3D steel frame (substructure)</td>
</tr>
<tr>
<td>Bennetts et al. (1985)</td>
<td>Australia</td>
<td>Localized fire triggered by car</td>
<td>two-floor open-deck car park</td>
</tr>
<tr>
<td>Rubert and Schaumann (1986)</td>
<td>Germany</td>
<td>Electrical furnace fire</td>
<td>quarter-to-half frame (substructure)</td>
</tr>
<tr>
<td>Latham et al. (1987)</td>
<td>UK</td>
<td>Compartment fire</td>
<td>One-storey compartment replicating an office in a multi-storey building or a six-bed hospital ward</td>
</tr>
<tr>
<td>Cooke and Latham (1987)</td>
<td>UK</td>
<td>Natural fire using wooden cribs</td>
<td>rugby post frame (substructure)</td>
</tr>
<tr>
<td>Li et al. (1999)</td>
<td>China</td>
<td>Gas furnace fire</td>
<td>small scale quarter rigid steel frames (2 bay 1 storey substructure)</td>
</tr>
<tr>
<td>CTICM (2000)</td>
<td>France</td>
<td>Localized fire triggered by car</td>
<td>two-storey braced composite car park</td>
</tr>
<tr>
<td>Wald et al. (2009)</td>
<td>Czech</td>
<td>Localized and Compartment fire</td>
<td>Abandoned 3-storey steel frame building</td>
</tr>
<tr>
<td>Dong and Prasad (2009)</td>
<td>China</td>
<td>Large scale furnace compartment fire</td>
<td>two-storey, two bay composite steel frame</td>
</tr>
</tbody>
</table>
2.4 Discussion and conclusions

From the literature review considering both fire engineering and robustness design, it is found that the majority of the work was focused separately on each aspect, whereas investigations that combine the two are rare.

On one hand, emphasis is given to the fire resistance of individual components in current fire design practice. Although sub-structural models and advanced global models are mentioned in design codes, the treatment of ductility, redundancy, and continuity of structures is not adequately addressed. Of course a detailed numerical model of a whole structure or sub-structure considering ductility supply can be established in accordance with the current design codes to assess the vulnerability of structures to progressive collapse under well-defined fire scenarios; however, this approach can be too computationally demanding for design-oriented practice, and does not comply with typical robustness provisions, which are intended to limit the progression of local damage under unforeseen events.

On the other hand, fire scenarios are not currently considered in the current specialist design codes for robustness assessment, where the scenario of sudden column loss is normally assumed. Although this assumption is verified to be capable of providing an upper bound on the structural deformation demands (Gudmundsson and Izzuddin, 2010) considering column damage by blast, it is not immediately applicable to fire conditions. This is because of the difference of the inherent mechanism between the scenarios of fire induced column buckling and idealised sudden column loss (i.e. different dynamic effects induced, different residual resistance of the directly affected column, diverse response of the fire floor and the ambient floors, unexpected damages caused by thermal stress and uneven temperature field, complex joint response in fire, etc.). Although Wang and Wald (2008) suggested several possible solutions for enhancing structural robustness under fire conditions, these are largely prescriptive recommendations and do not offer a quantifiable framework. Some other researchers (Liew, 2008; Quiel and Marjanishvili, 2010) also investigated the robustness of structures subject to fire, but the fire was considered to occur subsequent to blast loading that causes severe damage to the structure before the fire. Therefore, a wide knowledge gap still exists for the consideration of localised fire hazard in the design-oriented progressive collapse
assessment. The remaining chapters of this thesis are devoted to offer a comprehensive design framework to narrow this gap.
CHAPTER 3

Individual Member Behaviour in Fire

3.1 Introduction

This chapter investigates the ambient and fire behaviour of individual structural members, including steel and composite beams, steel columns, and composite slabs. Two main objectives are identified for this study. The first one is to gain sufficient confidence of ADAPTIC (Izzuddin, 1991) models in simulating the individual members under elevated temperatures through a series of benchmark studies, where the ADAPTIC predictions are compared with the available experimental data or numerical predictions conducted by other researchers. Provided that each individual model can be simulated with sufficient accuracy, the reliability of more complex structural models can be assumed. The second objective of this study is to acquire the influences of various parameters on the response of the individual members affected by fire. This can offer a clearer understanding of the fire behaviour of individual members to be employed in more complex structural or sub-structural models comprising individual beams, columns and joint components.

In the following, benchmark studies on steel beams, composite beams, steel columns, and composite slabs are presented in Sections 3.2-3.5, respectively. Related parametric studies are further conducted to explore the behaviour of these members with different geometric, loading (either mechanical or thermal) and boundary conditions. Due to the special role that joints can play in determining the structural ductility and thus developing failure criteria, investigations on joints are exclusively discussed later in Chapter 4.
3.2 Steel beams

The fire tests on steel beams conducted by Liu et al. (2002) in the Manchester Fire Laboratory are employed in this study to verify the accuracy of the beam model established in ADAPTIC. Once sufficiency is established in the numerical model, further investigations on the tested beams are conducted through a series of parametric studies, which consider the influences of thermal effects, temperature gradients, axial end restraints and rotational end restraints.

3.2.1 Benchmark study on steel beams

The tested specimen was a two-metre long S275 178×102×19 universal beam that was connected to two fire protected 152×152×30 universal columns (similar to a ‘rugby goal post’). Flush end-plate joints were used to connect the beam and the two columns. Three different values of axial restraint were considered, which are 8kN/mm, 35kN/mm and 62kN/mm, where the value of 8kN/mm comes directly from the transverse bending stiffness of the supporting columns, and the other two restraint values are realized by using additional struts between the columns and the reaction frame. Three main levels of design load ratios were employed (0.3, 0.5 and 0.7). The arrangement of the experiment facility is illustrated in Figure 3.1.

![Fig. 3.1 Illustration of experimental facility (Liu et al., 2002)](image)

During the heating procedure, the top flange of the tested beam was protected by a 15mm thick ceramic fire blanket in order to simulate the heat-sink effect caused by concrete slab (Liu et al., 2002). In addition, 50mm thick ceramic fire blankets were
applied on both the columns and the joints to ensure that these structural parts are minimally affected by the fire. The time versus temperature curves of the top flange, the web and the bottom flange of the tested beam are given in Figure 3.2.

![Temperature distribution over steel beam depth during heating](image)

**Fig. 3.2** Temperature distribution over steel beam depth during heating (Liu *et al.*, 2002)

Among the sixteen tests conducted in the Manchester Fire Laboratory, the tested beams with end-plate joints under typical load ratios (0.5 and 0.7) and typical axial restraints (8kN/mm and 62kN/mm) were selected by Liu *et al.* (2002) and Yin and Wang (2004) for numerical validations. The finite element software packages ABAQUS and FEAST were employed, where shell elements were used to model the beam flange and web. In ADAPTIC, the selected beam tests are simulated with simple beam-column elements. The elliptical material model (Song *et al.*, 2000), as illustrated in Figure 3.3, is used for steel with an ambient yield/proportional strength of 275MPa and elastic modulus of 210000MPa. The material degradation factors at elevated temperatures are based on the recommendations specified in Eurocode 3 (2005b). For the extended end-plate beam-column connections, a rotational stiffness of 14000 kNm/rad is assumed according to the corresponding analysis results (Yin and Wang, 2004). The nonlinear temperature distribution of the tested beam is considered as a quadratic/parabolic curve \( T(y) = ay^2 + by + c \) over the cross-section in ADAPTIC, as illustrated in Figure 3.2. Two structural responses are considered to compare with the experimental results, which are the temperature-deflection relationship and the temperature-axial compression relationship, where the results are shown in Figures 3.4 and 3.5 respectively.
Fig. 3.3 Illustration of elliptical model for steel

Fig. 3.4 Temperature-deflection relationships for tested steel beam (cont’d…)
Fig. 3.4 (… cont’d) Temperature-deflection relationships for tested steel beam

Fig. 3.5 Temperature-axial compression relationships for tested steel beam (cont’d…)
Fig. 3.5 (… cont’d) Temperature-axial compression relationships for tested steel beam
It is observed that the results predicted by the beam-column elements in ADAPTIC compare well with the test results and the numerical results conducted by other researchers using shell elements. A certain level of discrepancy is found, which can be attributed to the following reasons. Firstly, the temperature conditions are extracted from the test data at every five minutes for the ADAPTIC model, and a linear interpolation is adopted between the five-minute intervals. Secondly, the joints are assumed to be at ambient temperature at all times, but in the real test, the maximum temperature of the joints was 250 °C, which could affect the rotational stiffness of the joints. Thirdly, all the temperature information is based on one typical temperature distribution curve provided by Liu et al. (2002), while it is unlikely that the temperature conditions in every test follow exactly the same temperature-time relationship. Finally, as can be clearly seen in the temperature vs. axial compression relationships for the axial restraint of 62kN/mm, slower initial increasing rates in the compressive internal force are predicted by the numerical results. This is due to the fact that the stiffness of struts which provide additional axial restraints cannot be fully utilised in the initial stages, as explained by Yin and Wang (2004) who conducted the test. Despite the minor discrepancies, the accuracy of the ADAPTIC beam model is still shown to be good.

3.2.2 Further discussions on steel beams

3.2.2.1 Effect of thermal expansion

For steel or composite frames, high levels of axial restraints provided by the surrounding cooler structures are normally applied to the fire affected steel beams. Due to the thermal expansion effects, early failure (e.g. flexural buckling) of the affected steel beam can be triggered at a relatively lower temperature. Fire tests such as the Cardington experiment also highlight the importance of thermal expansion on the structural behaviour under fire (Elghazouli et al., 2000). In this context, the influence of thermal expansion on the behaviour of the tested beam with a load ratio of 0.5 is considered in this section. Figures 3.6 and 3.7 present respectively the mid-span deflection vs. temperature and the axial compressive internal force vs. temperature for the beam under the two levels of axial restraints (8kN/mm and 62kN/mm), with and without considering thermal expansion.
Fig. 3.6 Effect of thermal expansion on beam deflection

Fig. 3.7 Effect of thermal expansion on beam axial internal force

It can be observed that thermal expansion has an evident influence on the behaviour of the axially restrained beams under fire. As shown in Figure 3.6, the beam subject to either the 8kN/mm or the 62kN/mm axial restraint without thermal expansion undergoes a similar response, where negligible deflections are found before the bottom flange temperature of 600°C. In contrast, a more considerable deflection is observed for the beam with thermal expansion under the same temperature. This is due to the fact that for the beam excluding thermal expansion, the limited initial deflection is only caused by
the degradation of material properties; whereas for the thermally expanded beam, a combined effect of material degradation, thermal bowing and the additional compressive internal force caused by the restraints against thermal expansion is induced, which can greatly increase the deflection. Considering the beam with thermal expansion, it is found that the level of the axial restraint has negligible influence on the deflection at lower temperatures, i.e., below 500°C, but as the temperature increases further, a greater deformation are induced earlier under increased axial restraint. As shown in Figure 3.6, the beam with the 62kN/mm axial restraint starts to deflect significantly at 600°C; this failure temperature can be delayed to more than 700°C when a reduced 8kN/mm restraint is applied. This implies that at the initial stages, the deflection is mainly governed by thermal bowing, but as the temperature keeps increasing, flexural buckling may be more dominant in determining the beam deflections.

3.2.2.2 Effect of temperature distribution over beam depth

Three temperature distributions over the beam depth are considered to investigate their influence on the fire behaviour of the steel beam, namely, real nonlinear distribution (quadratic), simplified linear distribution, and idealised uniform distribution, as illustrated in Figure 3.8. For the simplified linear distribution, the temperature is decreased linearly from the bottom to the top, while for the uniform distribution, no temperature gradient is considered. For all three cases, identical values of the bottom flange temperature are assumed, where for the ‘nonlinear’ and ‘linear’ cases, identical top flange temperatures are considered. Employing the three temperature distributions, Figures 3.9 and 3.10 respectively provide the bottom flange temperature vs. deflection and the bottom flange temperature vs. axial internal force curves for the beam, where two load ratios (0.3 and 0.7) are considered and the value of axial restraint for the beam is 35kN/mm.

![Cross-section](image)

**Fig. 3.8** Temperature distributions over beam depth
It can be seen that before the ‘initial failure temperature’ (where the deflection starts to increase abruptly), the deflections of the beam with the nonlinear temperature distribution and the simplified linear temperature distribution are similar, while the beam with the uniform temperature distribution experiences a much smaller deflection. This again confirms that thermal bowing has a significant effect on beam deflections under a relatively low temperature. As the temperature increases beyond the initial
failure temperature, the deflection of the beam under the uniform temperature distribution exceeds that of the beam under the other two temperature distributions. This is attributed to an overestimation of the top flange temperature when a uniform temperature distribution is considered, which leads to larger internal compressive forces due to thermal expansion (as clearly shown in Figure 3.10). Therefore, a lower temperature resistance of the beam is found due to earlier buckling. As a general remark of the three different temperature distributions, the assumption of the simplified linear temperature distribution can predict the behaviour of beams at elevated temperature reasonably well, although it is likely to overestimate the critical temperature. On the other hand, although the assumption of the uniform temperature distribution renders more conservative results, due to its simplicity, it is still acceptable (e.g. in preliminary designs) when real temperature distributions are not available.

3.2.2.3 Effect of axial restraint stiffness

This section considers various levels of linear axial restraints applied at the ends of the tested beam, namely, 0.01$K_a$, 0.1 $K_a$, 0.3 $K_a$, 0.6 $K_a$, 1.0 $K_a$, 2.0 $K_a$ and rigid axial restraint, where $K_a$ is the axial stiffness of the beam at ambient temperature: $K_a = E A / L = 255\text{kN/mm}$ ($E$ is young’s modulus, $A$ is cross-sectional area, and $L$ is beam length). The results of the temperature vs. deflection and the temperature vs. axial force are depicted in Figures 3.11 and 3.12, respectively.

![Fig. 3.11 Effect of axial restraint stiffness on beam deflection: a) L.R. = 0.3, b) L.R. = 0.7](image)
Before the critical temperature at which the beam deflection starts to increase abruptly, the development of the beam deflection varies with the different applied end axial restraints. Generally, larger deflections can be found in the beam with larger axial restraint stiffness, which is due to the generation of greater compressive axial forces. Accordingly, the critical temperature is decreased as the stiffness of the axial restraint is increased. Moreover, the compressive force starts to decrease at the critical temperature, as shown in Figure 3.12. As the beam enters into a further stage, the catenary stage, the deflection increment of the beam with larger axial restraints starts to slow down and finally the deflection becomes smaller than that of the beam with less rigid axial restraints. This phenomenon indicates that more evident catenary action can be developed with the provision of stiffer axial restraints. It should be noted that although catenary action is beneficial for enhancing the fire resistance of beams at large deflections, it inevitably overburdens the surrounding substructures and joints, thus the advantage of catenary action can only be relied upon provided that the surrounding structure is carefully designed with sufficient resistance.

3.2.2.4 Effect of rotational restraint stiffness

Rotational stiffness provided by the connected joints and columns varies with their type and geometric property. In general, extended or flushed end-plate joints can be seen as rotationally rigid or semi-rigid, while web cleat joints are usually treated as a pin joint
with negligible rotational stiffness. In order to check the influence of rotational restraint on the behaviour of the heated beam, six values of end rotational restraint are considered, which are zero stiffness, \(0.05Ei/L\), \(0.2Ei/L\), \(0.5Ei/L\), \(1.0Ei/L\), and rigid rotational stiffness, where \(E\) is the young’s modulus, \(I\) is the second moment of area of the beam, and \(L\) is the beam length.

Figures 3.13 and 3.14 respectively show the temperature vs. deflection and the temperature vs. axial internal force for the beam subject to different rotational restraints with the axial restraint stiffness of 35kN/mm. Clearly, the deflection of the beam with a stiffer rotational restraint is smaller than that with a lower level of rotational restraints. In addition, when the level of the rotational restraint exceeds a certain value (i.e. for the cases of \(0.5Ei/L\), \(1.0Ei/L\), and rigid rotational stiffness), the influence of the rotational restraints on the deflection is limited. With respect to the response of the internal axial force, the influence of the rotational restraints is much less than that of the axial restraints, and in general, the internal compressive force starts to decrease when an increase of the beam deflection is observed.

![Fig. 3.13 Effect of rotational restraint stiffness on beam deflection: a) L.R. = 0.3, b) L.R. = 0.7](image-url)
3.3 Composite beams

This section utilises ADAPTIC (Izzuddin, 1991) beam-column models for simulating composite beams under both ambient and elevated temperature scenarios. For the ambient temperature case, two simply-supported composite beam tests named ‘test A3’ and ‘test A5’ conducted by Chapman and Balakrishnan (1964) are selected. For the elevated temperature conditions, two fire tests on simply-supported composite beams named ‘test 15’ and ‘test 16’ conducted by Wainman and Kirby (1988) are referred to. These tests have been simulated by Huang et al. (1999) using the finite element program VULCAN. Recently, Haremza et al. (2009) also modelled these tests using the commercial FE software package ABAQUS. In this study, the composite beam models established in ADAPTIC are compared with the test results and the numerical predictions conducted by the other researchers. The ADAPTIC model considers the composite beams with a full concrete slab-steel interaction, since from the numerical predictions by Haremza et al. (2009), it was shown that the responses of the composite beams with partial and full interactions are similar.

3.3.1 Benchmark study on ambient composite beams

Two simply-supported beams, loaded with a central point load until failure, are considered, as shown in Figure 3.15. In ADAPTIC, beam-column elements are
employed for modelling the steel beam and the concrete flange, which are connected by rigid links representing full interaction between the steel and concrete, as illustrated in Figure 3.16.

![Diagram of composite beam model in ADAPTIC](image)

**Fig. 3.15** Dimensions of tested composite beam (Huang *et al.* 1999)

**Fig. 3.16** Composite beam model in ADAPTIC

According to the material properties for the tests provided by Huang *et al.* (1999) and Haremza *et al.* (2009), the corresponding stress-strain relationships for steel and concrete employed in ADAPTIC are illustrated in Figure 3.17a and 3.17b, respectively. For the steel beam and the reinforcement, bilinear elasto-plastic models with kinematic strain hardenings are adopted. The material properties for steel are listed in Table 3.1, where $f_y$ is the yield stress, and $E$ is the modulus of elasticity. For concrete, a bilinear model is employed for tension, accounting for softening after cracking, while for compression, two alternative stress-strain relationships are considered for the ascending branch, which are ‘quadratic’ and ‘linear’ responses. For the quadratic compressive response, the initial tangent stiffness is taken as equal to the initial tensile stiffness. The details for concrete properties are listed in Table 3.2, where $f_c$ is the compressive strength of concrete, $E_{c1}$ is the initial tangent modulus of concrete in compression for the quadratic response, $\varepsilon_c$ is the compressive strain in concrete at $f_c$, $f_t$ is the tensile strength of concrete, $\varepsilon_{t1}$ is the tensile strain in the concrete at $f_t$, and $\varepsilon_{t2}$ is the ultimate tensile strain in concrete.

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Steel</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Stress ($f_y$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity ($E$)</td>
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<td></td>
</tr>
<tr>
<td>Compressive Strength ($f_c$)</td>
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<td></td>
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<td>Initial Tangent Modulus in Compression ($E_{c1}$)</td>
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<td>Compressive Strain at $f_c$ ($\varepsilon_c$)</td>
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</tr>
<tr>
<td>Tensile Strength ($f_t$)</td>
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<td></td>
</tr>
<tr>
<td>Tensile Strain at $f_t$ ($\varepsilon_{t1}$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate Tensile Strain in Concrete ($\varepsilon_{t2}$)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig. 3.17 Stress-strain behaviour of materials: a) steel, b) concrete

<table>
<thead>
<tr>
<th>Test name</th>
<th>( f_y ) (MPa)</th>
<th>( E ) (MPa)</th>
<th>Strain hardening factor ( \mu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test A3 beam</td>
<td>302</td>
<td>210000</td>
<td>0.48%</td>
</tr>
<tr>
<td>Test A5 beam</td>
<td>290</td>
<td>210000</td>
<td>0.46%</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>600</td>
<td>210000</td>
<td>0.97%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test name</th>
<th>( f_c ) (MPa)</th>
<th>( E_c ) (MPa)</th>
<th>( \varepsilon_c )</th>
<th>( f_t ) (MPa)</th>
<th>( \varepsilon_{t1} )</th>
<th>( \varepsilon_{t2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test A3</td>
<td>35</td>
<td>27000</td>
<td>0.0021</td>
<td>2.7</td>
<td>0.0001</td>
<td>0.001</td>
</tr>
<tr>
<td>Test A5</td>
<td>51</td>
<td>37000</td>
<td>0.0024</td>
<td>3.7</td>
<td>0.0001</td>
<td>0.001</td>
</tr>
</tbody>
</table>

The mid-span deflection vs. load response for the tested beams obtained from ADAPTIC are compared with those from VULCAN, ABAQUS as well as the test results, as shown in Figures 3.18 and 3.19 for tests A3 and A5 respectively. Good correlations are observed between the ADAPTIC results and the other predictions. Moreover, the composite beam employing the concrete model with the idealised linear initial compressive stiffness is found to yield a slightly larger deflection than that with quadratic concrete model under the same load, which indicates that the idealised linear initial compressive stress-strain relationship of concrete can lead to conservative results when predicting the deflection of the composite beam.
3.3.2 Benchmark study on composite beams under fire

Two of the fire tests conducted by Wainman and Kirby (1988), named ‘test 15’ and ‘test 16’, are chosen for the benchmark study on composite beams at elevated temperature. The aim of the tests was to investigate the fire behaviour of the simply-supported composite beams subject to a standard fire ISO834. Four point loads were applied along the length of each beam, and load ratios of 0.294 and 0.564 were considered for the tests 15 and 16 respectively, corresponding to point loads of 32.47kN and 62.36kN. A UB 254×146×43 steel beam was adopted and a 130mm thick reinforced...
slab was connected to the steel beam with 32 shear studs along the length. The details of the tested beam are illustrated in Figure 3.20. Full interaction between concrete and steel is assumed in the present study.

![Figure 3.20 Dimensions of tested composite beams in fire, Huang et al. (1999)](image)

Similar to the beam in the previous section, the ambient properties for the steel/reinforcement and the concrete are listed in Tables 3.3 and 3.4.

**Table 3.3 Ambient material properties for steel – tests 15 and 16**

<table>
<thead>
<tr>
<th>Material</th>
<th>$f_y$ (MPa)</th>
<th>$f_p$ (MPa)</th>
<th>$E_s$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam steel</td>
<td>255</td>
<td>255</td>
<td>210000</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>600</td>
<td>600</td>
<td>210000</td>
</tr>
</tbody>
</table>

**Table 3.4 Ambient material properties for concrete – tests 15 and 16**

<table>
<thead>
<tr>
<th>Material</th>
<th>$f_{c1}$ (MPa)</th>
<th>$\varepsilon_{c1}$</th>
<th>$\varepsilon_{c2}$</th>
<th>$f_{t1}$ (MPa)</th>
<th>$\varepsilon_{t1}$</th>
<th>$\varepsilon_{t2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>30</td>
<td>0.0025</td>
<td>0.02</td>
<td>2.9</td>
<td>0.00025</td>
<td>0.002</td>
</tr>
</tbody>
</table>

During the heating process, the temperature distribution over the beam depth is given in Figures 3.21 and 3.22 for the steel beam and concrete flange, respectively. No temperature variation is considered along the length of the composite beam. Similar to the bare steel beam considered in Section 3.2, the elliptical models is employed for steel at elevated temperature. For concrete, a nonlinear material model is employed, which accounts for elevated temperature effects and compressive nonlinearity. The equation for the ascending compressive branch is expressed by

$$\sigma / f_c = 3 (\varepsilon_c / \varepsilon_{c1}) / \left(2 + (\varepsilon / \varepsilon_{c1})^3\right).$$

The material degradation properties for steel and concrete in ADAPTIC are based on the recommendations specified in Eurocode 3 (2005b) and Eurocode 4 (2005) respectively, as illustrated in Figure 3.23.
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Fig 3.21 Temperature distribution over steel beam depth during heating (Huang et al. 1999)

Fig 3.22 Temperature distribution over concrete depth during heating (Huang et al. 1999)
Fig. 3.23 Steel and concrete models for ambient and elevated temperatures

The deflection-temperature curves obtained from ADAPTIC, ABAQUS, VULCAN and the experimental results for tests 15 and 16 are given in Figures 3.24 and 3.25 respectively. Good correlation is found between the ADAPTIC results and the other numerical predictions, but small discrepancies generally exist between the numerical and test results, particularly for test 16. This is attributed to the difficulties during the tests in producing a prefect simple support condition in a furnace at high temperatures, as explained by Huang et al. (1999).

Fig. 3.24 Temperature-deflection response for test 15
3.4 Steel columns

With the aim of validating the capability of the ADAPTIC beam-column elements in predicting the behaviour of steel columns subject to fire, this section compares column models built in ADAPTIC with a real column fire test (Tan et al. 2007). Related FE models established by other researchers (Huang and Tan, 2007) are also referred to. This is followed by a further discussion on various parameters that can potentially influence column response under elevated temperature.

3.4.1 Benchmark study on steel columns

The considered fire test was conducted by Tan et al. (2007), where the main aim was to investigate the elevated temperature response of axially restrained columns with various slenderness ratios. All test specimens had an effective length of 1.74m that was comprised of a 1.5m-length of the column itself and two knuckle bearings of 0.12m-length at both ends of the column to provide pinned end connections. The temperature distributions across the cross-section and along the column length were monitored throughout the heating procedure, as shown in Figure 3.26. It can be seen that the temperature is generally uniform along the length of the column, but it tends to decrease towards the two ends due to heat loss.
In ADAPTIC, elasto-plastic beam-column elements and two rigid links are employed to simulate the tested column and the two end knuckle bearings, respectively, as illustrated in Figure 3.27. Three values of initial column mid-height imperfections with respect to the weak axis (0mm, 1mm and 2mm) are employed. Although the two ends of the tested column were assumed as pinned in the experiment, inevitable end rotational friction due to bearing was considered by Tan et al. (2007). A friction coefficient $\mu_u = 0.2$ is taken, the equivalent rotational stiffness due to friction is computed as $K_r = M_u / \theta_u$, where $M_u$ is the ultimate restoring moment at the two ends, and $\theta_u$ is the tested end rotation just before the buckling of the columns. The stiffness of the equivalent rotational restraints applied at the column ends for all the specimens can be found in Huang and Tan (2007), and these values are also adopted in the ADAPTIC models. Two sets of tests, SR45 and
SR97, are selected for the current benchmark study. The test code ‘SR’ is short for ‘slenderness ratio’ and the number followed (‘45’ or ‘97’) is the value of the slenderness ratio. Ambient characteristics (elastic modulus $E_0$, yield strength $f_y$, applied load $P_0$, stiffness of axial restraint $K_l$ and stiffness of rotational restraint $K_r$) of each model are listed in Table 3.5. The elliptical model for steel at elevated temperature is used, and the material degradation properties are based on the recommendations specified in Eurocode 3 (2005b).

### Table 3.5 Properties of column models

<table>
<thead>
<tr>
<th>Test name</th>
<th>$E_0$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$P_0$ (kN)</th>
<th>$K_l$ (kN/mm)</th>
<th>$K_r$ (kN.m/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SR45-1</td>
<td>201</td>
<td>326</td>
<td>708.5</td>
<td>0.00</td>
<td>142.3</td>
</tr>
<tr>
<td>SR45-2</td>
<td>201</td>
<td>326</td>
<td>708.5</td>
<td>33.38</td>
<td>142.3</td>
</tr>
<tr>
<td>SR45-3</td>
<td>201</td>
<td>326</td>
<td>702.7</td>
<td>41.02</td>
<td>141.1</td>
</tr>
<tr>
<td>SR97-1</td>
<td>200</td>
<td>320</td>
<td>134.0</td>
<td>0.00</td>
<td>31.3</td>
</tr>
<tr>
<td>SR97-2</td>
<td>200</td>
<td>320</td>
<td>133.9</td>
<td>14.70</td>
<td>31.3</td>
</tr>
<tr>
<td>SR97-3</td>
<td>200</td>
<td>316</td>
<td>179.0</td>
<td>21.62</td>
<td>41.8</td>
</tr>
<tr>
<td>SR97-4</td>
<td>200</td>
<td>316</td>
<td>136.1</td>
<td>30.11</td>
<td>31.7</td>
</tr>
</tbody>
</table>

The results derived from the ADAPTIC models are compared with the test results as well as the corresponding numerical results obtained from Huang and Tan (2007) using the finite element programme FEMFAN2D (Tan et al., 2002). Figures 3.28 – 3.34 give the numerical and the test predictions for the seven considered columns, where the variation of the mid-height transverse displacement and the internal axial force with time are provided for each tested column. The column internal force is represented by the ratio of the elevated temperature axial force over that under the initial ambient condition.

In general, the ADAPTIC predictions agree adequately with the test and FEMFAN2D results. It can be observed that the initial imperfections have an evident influence on the column response, and the failure time of the column is greatly reduced in the presence of such imperfections. In some cases discrepancies are found, especially for the SR45-1 and SR45-2 columns where the failure times obtained in the tests are underestimated. This may due to an underestimation of the equivalent end rotational stiffness $K_r$, that can delay the column buckling time. Moreover, a typical temperature distribution in the
heated columns is employed in all the models, which can also cause discrepancies since the temperature distribution in different column tests can slightly vary.

Fig. 3.28 Response of SR45-1 column: a) mid-height displacement, b) axial force

Fig. 3.29 Response of SR45-2 column: a) mid-height displacement, b) axial force
Fig. 3.30 Response of SR45-3 column: a) mid-height displacement, b) axial force

Fig. 3.31 Response of SR97-1 column: a) mid-height displacement, b) axial force
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Fig. 3.32 Response of SR97-2 column: a) mid-height displacement, b) axial force

Fig. 3.33 Response of SR97-3 column: a) mid-height displacement, b) axial force
Fig. 3.34 Response of SR97-4 column: a) mid-height displacement, b) axial force

Table 3.6 Column failure time obtained in tests and predicted numerically (minutes)

<table>
<thead>
<tr>
<th>Test name</th>
<th>Test</th>
<th>FEMFAN2D</th>
<th>ADAPTIC, Imp.=0mm*</th>
<th>ADAPTIC, Imp.=1mm*</th>
<th>ADAPTIC, Imp.=2mm*</th>
</tr>
</thead>
<tbody>
<tr>
<td>SR45-1</td>
<td>78.4</td>
<td>74.7</td>
<td>72.2</td>
<td>71.1</td>
<td>70.1</td>
</tr>
<tr>
<td>SR45-2</td>
<td>78.1</td>
<td>67.5</td>
<td>66.2</td>
<td>64.3</td>
<td>64.3</td>
</tr>
<tr>
<td>SR45-3</td>
<td>59.9</td>
<td>62.1</td>
<td>66.2</td>
<td>65.1</td>
<td>64.2</td>
</tr>
<tr>
<td>SR97-1</td>
<td>73.2</td>
<td>79.3</td>
<td>75.3</td>
<td>72.3</td>
<td>71.1</td>
</tr>
<tr>
<td>SR97-2</td>
<td>47.4</td>
<td>46.8</td>
<td>50.2</td>
<td>46.2</td>
<td>44.3</td>
</tr>
<tr>
<td>SR97-3</td>
<td>34.0</td>
<td>33.6</td>
<td>39.0</td>
<td>36.3</td>
<td>34.1</td>
</tr>
<tr>
<td>SR97-4</td>
<td>38.7</td>
<td>41.1</td>
<td>45.0</td>
<td>43.5</td>
<td>43.1</td>
</tr>
</tbody>
</table>

* Imp. represents imperfection

Table 3.6 summarises the ‘failure times’ of the columns predicted by the numerical studies and obtained in the tests. The failure time is defined in a conventional way when the column internal axial force returns back to the initial ambient value. Judging from Figures 3.28 – 3.34, the conventional way to define ‘failure time’ appears to be realistic for most of the isolated column tests. Due to the limit of the experimental facility, the post-buckling response of the specimens was not fully recorded. Further discussions in the following study will consider both the buckling and post-buckling behaviour of the restrained column as well as other factors that can influence the column behaviour under fire.
3.4.2 Parametric studies on steel columns

For a building under real fire situations, a pronounced interaction between the heated column and the adjacent cooler members is expected. This section provides a systematic analysis on the column models subject to various boundary conditions and loading conditions. The parametric study considers the influence of thermal expansion, axial end restraints, rotational end restraints, and load ratios. Furthermore, the previously used test programme (Tan et al., 2007) did not consider the post-buckling behaviour of the heated columns, and this will be investigated in the following parametric study. S355 steel columns with a 3-metre length and an initial imperfection of 1mm (1/3000 of the length) are assumed for this study. Different columns sizes will be used in order to consider various values of slenderness ratio. The load ratio is defined as the ratio of the applied axial load over the buckling resistance of the columns specified in Eurocode 3 (2005c). When calculating the axial load capacity of each column, it is assumed that the columns are pin-ended (effective length = 3m). A uniform temperature distribution is assumed along the length and across the cross-section.

3.4.2.1 Effect of thermal expansion

UC 254×254×132 (slenderness ratio=45) and UB 254×102×28 (slenderness ratio=135) columns are considered here. Two levels of load ratio (0.3 and 0.7) are applied, where the buckling resistance for the two columns according to Eurocode 3 (2005c) are 4838.4kN and 339.3kN, respectively. Two axial restraint conditions are considered: no axial restraint and 0.2K_a axial restraint. The values of the axial stiffness K_a for the two types of columns are 1176kN/mm and 252.7kN/mm respectively. Zero end rotational stiffness is assumed for all the column models. Figures 3.35 and 3.36 provide the responses (mid-height displacement and axial force) of the columns with the slenderness ratios of 45 and 135 respectively, where ‘LR’ represents load ratio, ‘AS’ represents axial stiffness, ‘WT’ indicates ‘with thermal expansion’ and ‘NT’ indicates ‘no thermal expansion’.

The results show that ignoring thermal expansion has negligible influence on the axially unrestrained columns because no additional thermal compression is induced. However, significant influences are found in the axially restrained columns if thermal expansion is excluded. It is observed that thermal expansion in the axially restrained columns can greatly reduce the column buckling temperatures. This results from the additional
thermal compressive force induced by axial restraints, as observed in Figure 3.36, which shows that that the additional thermally induced compressive force can be as large as 2.5 times the initial ambient compressive force. This study highlights the importance of the role played by thermal expansion in the elevated temperature response of columns.

![Graph showing mid-height displacement and F/F₀ against temperature](image)

**Fig. 3.35** Effect of thermal expansion on UC 254×254×132 column (slenderness ratio = 45):

a) mid-height displacement, (b) $F/F₀$
3.4.2.2 Effect of axial restraint

In the current codes for column design at elevated temperature, the conditions of end restraints, especially axial restraints, are not comprehensively considered. This may introduce a pronounced inaccuracy in predicting the fire resistance of a column with axial restraint which can greatly reduce the buckling temperature. In addition, the P-Δ effects can be amplified by initial imperfections due to thermal stress. On the other hand, the existence of axial restraint may conversely enhance the resistance of a heated
column due to upward pulling forces during the post buckling stage. This phenomenon has attracted recent attention particularly for the consideration of structural robustness. This section presents the response of a 3-metre long UC 254×254×132 column subject to different levels of axial restraint, namely, no axial restraint, 0.02EA/L, 0.05 EA/L, 0.1EA/L, 0.2EA/L, 1.0EA/L, and infinite axial restraint, where EA/L is the elastic axial stiffness of the column (1176kN/mm for UC 254×254×132).

![Graph 1](image1.png)

(a)

![Graph 2](image2.png)

(b)

**Fig. 3.37** Effect of axial restraint on UC 254×254×132 column under load ratio of 0.3: (a) mid-height displacement, (b) $F/F_0$
Fig. 3.37 and 3.38 present the variation of the mid-height transverse displacement and the normalised axial force with temperature for the UC 254×254×132 column subject to different levels of axial restraints under the two load ratios of 0.3 and 0.7, respectively. It is evident that larger axial restraint stiffness leads to earlier column buckling. The initial buckling temperatures for the axially unrestrained column under a load ratio of 0.3 and 0.7 are 630 °C and 400 °C, respectively. When a 0.1EA/L axial restraint is applied, the initial buckling temperatures under these two load ratios
decrease dramatically to 400 °C and 240 °C respectively. Further higher axial restraints delay the development of the column deflection in the post-buckling stage. Under a mid-height displacement of 200mm after column buckling, the corresponding temperature for the column under an axial restraint of more than 1.0EA/L can be as high as 900°C, which are considerably higher than the temperature for the unrestrained case. Therefore, the column under a significant axial restraint experiences a more gradual development of the lateral deflection with a more distinct post-buckling response, so a large axial restraint stiffness tends to slow down the column buckling process, thus avoiding a sudden failure of the column. However, in the context of a whole building structure, the associated reduction in the column axial force implies a redistribution to the floors above, which would have to provide sufficient strength and ductility.

3.4.2.3 Effect of rotational restraint

Considering the continuity in a building structure, rotational restraints may arise from the adjacent beams, slabs, and other columns connected to the column ends. The test results of Tan et al. (2007) indicated that end rotational restraints can bring significant benefits to the column resistance. To verify the influence of rotational restraint on the column behaviour, a 3-metre long UB 254×102×28 subject to different levels of rotational stiffness is considered, namely, no rotational restraint, 0.05EI/L, 0.2EI/L, 0.5EI/L, 1.0EI/L, and infinite rotational restraint, where E is Young’s modulus, I is the column’s second moment of area with respect to the weak axis, and L is the length of the column. The same load is considered for all columns, corresponding to a load factor of 0.5 based on the resistance of the pin-ended column. In most cases, rotational restraints are in conjunction with axial restraints, hence two levels of axial restraint stiffness are considered, namely, no axial restraint and 0.2EA/L.

Figures 3.39 and 3.40 show the variation of the mid-height displacement and normalised axial force with temperature for the UB 254×102×28 column subject to various levels of rotational restraint under the two considered axial restraint levels, respectively. It is observed that the presence of rotational restraint can indeed delay column buckling, and its influence on the column without axial restraint is relatively larger than that under the axial restraint of 0.2EA/L. For the axially unrestrained column with no rotational restraint, the failure temperature is approximately 550 °C, while for the column with full rotational restraint, the failure temperature is increased by 100 °C.
This is due to a reduced effective length of the column by the existence of end rotational stiffness, thus enhancing the buckling resistance.

Fig. 3.39 Effect of rotational restraint on UB254×102×28 column without axial restraints:
(a) mid-height displacement, (b) $F/F_0$
A similar improvement is observed for the column with an axial restraint stiffness of 0.2EA/L when the rotational restraint stiffness is increased. According to the conventional definition of ‘critical column failure time’, when the compressive axial internal force reduces back to the initial ambient value, the column’s critical failure temperature is also significantly enhanced with increasing rotational restraint stiffness, as shown in Figure 3.40 (b).
3.4.2.4 Effect of load ratio

Finally, the elevated temperature response of the 3m long UB 254×102×28 is investigated under different load ratios of 0.2, 0.4, 0.6, and 0.8. The compressive buckling stress of the column derived from Eurocode 3 (2005c) is 94MPa. Two axial restraint conditions are assumed, namely, no restraint and a restraint stiffness of 0.3EA/L. Figures 3.41 and 3.42 show the variation of the mid-height transverse displacement and the normalised axial force with temperature for the UB 254×102×28 column subject to various load ratios under the two axial restraint stiffness conditions. Clearly, a higher load ratio leads to a lower critical temperature, regardless of the conditions of axial restraints. For the column without axial restraint, the failure temperature under a load ratio of 0.8 is approximately 390°C, while for the smaller load ratio of 0.6 the failure temperature can be increased to around 510°C. Similarly, for the columns with an axial restraint stiffness of 0.3EA/L, the critical failure temperature for the column can be increased from less than 100°C to 500°C when the load ratio decreases from 0.8 to 0.2. This implies that to ensure a sufficient fire resistance, the columns could be designed more conservatively.

![Fig. 3.41 Effect of load ratio on UB254×102×28 column without axial restraint: a) mid-height displacement, (b) F/F₀](image)
3.5 Concrete and composite slabs

Concrete and composite steel-decked floor slabs can play a significant role in enhancing the robustness of steel/composite structures. In order to study the ambient and elevated temperature response of floor slabs and to validate the accuracy of slab models in ADAPTIC, a benchmark study is considered here, where the influence of slab profile, boundary conditions and the contribution of secondary beams are discussed. The corresponding responses predicted by ADAPTIC are compared with those obtained from other researchers (Gernay et al., 2009) who employed the computer program SAFIR (Franssen, 2005), a program developed for structural fire analysis at the University of Liège.

3.5.1 Slab model description

Two 16m×10m isolated slabs are considered in this study, one is a uniform thickness slab, and the other one is a ribbed slab. The basic profiles of the slab models are illustrated in Figure 3.43. The corresponding geometric dimensions are listed in Table 3.7, where \( a \) is the slab length, \( b \) is the slab width, \( t \) is the thickness of the steel deck, \( d \) is the depth for the uniform slab, \( s \) is the distance from the location of reinforcement to the top. In addition, for the slab with the ribbed profile, \( d_1 \) is the thickness of the cover, \( d_2 \) is the thickness of the rib, \( w_1 \) is the width of the rib bottom, and \( w_2 \) is the width of the
rib top. For both of the uniform thickness slab and the ribbed slab, the locations of reinforcement mesh are 50mm below the top face of the slab. The steel deck is assumed to be unidirectional for both models, which means that it only acts along the rib direction while no action is considered along the cross direction. Therefore, considering a steel deck thickness of 0.9mm, the equivalent area for steel deck can be modelled as 900mm²/m and 982mm²/m for uniform and ribbed slabs respectively, and 0mm²/m along the cross direction.

![Fig. 3.43 Geometric configuration of slab models, a) uniform slab, b) ribbed slab](image)

**Table 3.7** Details of slab dimensions (mm)

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>$a$</th>
<th>$b$</th>
<th>$d$</th>
<th>$d_1$</th>
<th>$d_2$</th>
<th>$w_{1}$</th>
<th>$w_{2}$</th>
<th>$w_{3}$</th>
<th>$t$</th>
<th>$s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
<td>16000</td>
<td>10000</td>
<td>100</td>
<td>70</td>
<td>60</td>
<td>272</td>
<td>376</td>
<td>224</td>
<td>0.9</td>
<td>50</td>
</tr>
</tbody>
</table>
In ADAPTIC, the new shell element developed by Izzuddin et al. (2004) for realistic modelling of composite and reinforced concrete floor slabs (considering the geometric orthotropy of the ribbed slab) subject to extreme loading conditions is employed. In SAFIR, the ribbed slab was modelled (Gernay et al., 2009) by uniform thickness shell elements of 70 mm height combined with concrete bars in the rib direction. The concrete bars were positioned at a distance of 98.4 mm from the top of the slab. Symmetry was taken into account and only a quarter of the slab was modelled in SAFIR.

Both ambient and elevated temperature conditions are considered in this study. For the ambient slab, a monotonically increased UDL (uniformly distributed load) is applied. With regard to the fire scenario, a constant uniform distributed load of 5kN/m² is applied onto the slab, and a monotonically increased temperature is subsequently applied up to a maximum value of 900°C on the bottom of the slab. A linear temperature distribution over the slab depth is assumed and is always in proportion during the heating process, as shown in Figure 3.44. It is noted that in SAFIR, the same temperature distribution as ADAPTIC is assumed for the uniform slab, while the temperature for the ribbed slab is slightly different due to different slab modelling strategies. The maximum temperature of the concrete bars in SAFIR is 886°C (assumed uniform over the bars modelling the rib), and the maximum temperature of the steel deck is 900°C, as shown in Figure 3.44(b).

---

**Fig. 3.44** Illustration of temperature gradients over slab depth, a) ADAPTIC; b) SAFIR (Gernay et al., 2009)
The ambient material properties of concrete and steel in ADAPTIC are listed in Tables 3.8 and 3.9 respectively. The material model for steel is the same as that employed for the previous studies on steel beams and columns (as illustrated in Figure 3.23). For concrete, an advanced nonlinear concrete model, which accounts for the combined effects of compressive nonlinearity, tensile cracking opening and closure, and elevated temperature, is employed. The concrete stress-strain relationship is illustrated in Figure 3.45, and details of this concrete model are elaborated in Izzuddin et al., (2004). With respect to the elevated temperature material properties, the temperature-dependent factors, e.g. degradation of strength and stiffness and thermal strain, are according to Eurocode 3 (2005b) and Eurocode 4 (2005) for steel and normal concrete respectively.

![Stress-strain relationships of concrete for slab model in ADAPTIC](image)

**Fig. 3.45** Stress-strain relationships of concrete for slab model in ADAPTIC

**Table 3.8** Ambient properties for concrete in ADAPTIC

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus E (N/mm²)</td>
<td>1.8×10⁴</td>
</tr>
<tr>
<td>Possion’s ratio</td>
<td>0.2</td>
</tr>
<tr>
<td>Tensile strength $f_t$ (N/mm²)</td>
<td>2.36</td>
</tr>
<tr>
<td>Compressive strength $f_c$ (N/mm²)</td>
<td>30</td>
</tr>
<tr>
<td>Compressive strain $\varepsilon_{c1}$</td>
<td>0.0033</td>
</tr>
<tr>
<td>Tensile softening slope $E_{cr}$ (N/mm²)</td>
<td>2716</td>
</tr>
<tr>
<td>Normalised compressive strength starting compressive nonlinearity $s_c$</td>
<td>0.4</td>
</tr>
<tr>
<td>Normalised residual compressive strength $r_c$</td>
<td>0.0</td>
</tr>
</tbody>
</table>
### Table 3.9 Ambient properties for steel in ADAPTIC

<table>
<thead>
<tr>
<th>Properties</th>
<th>Steel deck</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus E (N/mm²)</td>
<td>2.1×10⁵</td>
<td>2.1×10⁵</td>
</tr>
<tr>
<td>Yield strength f_y (N/mm²)</td>
<td>3.0×10²</td>
<td>6.0×10²</td>
</tr>
</tbody>
</table>

#### 3.5.2 Case description

Four cases are considered in this study, including one reference case and three comparison cases investigating the effect of slab ribs, planar restraints, and secondary beams. Both ambient and fire analyses are considered for all four cases. The details of these case models are presented next.

**Reference case**

The reference case is considered as the slab with the following properties:

- The slab is modelled as a uniform slab without the rib, and a value of \( d = d_1 + d_2 / 2 = 100 \) mm is assumed for the thickness of the reference slab.
- The slab is laterally restrained (in plane) and vertically supported along the four edges, while it is free to rotate at its boundaries.
- The secondary beams are not included.

The configuration of the reference model in ADAPTIC is shown in Figure 3.46.

![Fig. 3.46 Reference uniform thickness slab model established in ADAPTIC](image)

**Effect of slab ribs**

The composite slab with ribs is considered for comparison with the response of the uniform thickness slab (reference). The configuration of the ribbed slab model in ADAPTIC is shown in Figure 3.47.
Effect of planar restraints

The planar restraint is removed to allow the four edges of the slab freedom to move inwards. The edges for both the reference and the laterally unrestrained models are free to rotate. The aim of this study is to check whether the planar restraint has profound effects on the load resistance of the considered slab. Through the outcome of this study, the importance of the continuity of composite slabs and the stiffness of the supporting beams along the edges can be understood.

Effect of secondary beams

Finally, secondary beams are considered to investigate their influence on the load carrying capacities of the slab system, particularly under fire conditions. The modelling strategy for the shear interaction between the concrete slab and the steel beams is the same as that discussed in Section 3.3. Vertical and planar restraints are applied along the four edges of the slab as well as at the ends of the secondary beams (mid-depth). Under the elevated-temperature conditions, a uniformly distributed temperature over the steel beam depth is assumed, and the temperature of the steel beam is linearly increased to 900°C with the same rate of that in the bottom steel deck. Fully rigid shear connections are assumed between the slab and the secondary beams. The slab model with secondary beams is shown in Figure 3.48.
3.5.3 Results of slab benchmark study

Reference case

Figures 3.49 and 3.50 provide the load-deflection and the deflection-temperature response for the uniform thickness slab (reference case) under ambient and elevated temperature conditions, respectively. Favourable correlations are found between the ADAPTIC and SAFIR predictions. Due to concrete cracking, the ambient curves show a sudden increase in deflection at about 100mm. Subsequently, the resistance of the slab keeps increasing, where the ambient response of ADAPTIC slab model is slightly stiffer than the SAFIR model.

![Fig. 3.49 Ambient response of reference (uniform) slab](image1)

![Fig. 3.50 Elevated temperature response of reference (uniform) slab](image2)
CHAPTER 3 Individual Member Behaviour in Fire

**Effect of slab ribs**

Figures 3.51 and 3.52 provide the load-deflection response at ambient temperature and the deflection-temperature response at elevated temperature for the slab, where the uniform thickness slab (reference case) is compared with the ribbed slab. In general, good comparisons are found between the ADAPTIC and SAFIR predictions. A slightly greater discrepancy is observed for the ribbed slab at elevated temperature, which is mainly attributed to the different modelling approaches employed in the two programs for simulating the ribs.

![Fig. 3.51 Ambient response of uniform and ribbed slabs](image1)

![Fig. 3.52 Elevated temperature response of uniform and ribbed slabs](image2)
In addition, it can be seen the ribbed slab has a similar response to the uniform thickness slab with a depth of \( d_1 + d_2 / 2 \), which suggests that employing uniform slabs with equivalent depths can predict the response of ribbed slabs with desirable accuracy for the considered geometric configuration of the slab.

Effect of lateral restraints

Figures 3.53 and 3.54 provide the load-deflection response at ambient temperature and the deflection-temperature response at elevated temperature for the slab, where the slabs with and without planar restraint are compared. It can be observed that planar restraints have a profound effect on the behaviour of the slab. For the ambient condition, the slab with planar restraint can sustain twice the UDL of laterally unrestrained slab at a deflection of 200mm. At a further deflection of 600mm, the load resistance of the restrained slab is at least four times of that of the laterally unrestrained slab, which is mainly attributed to a more predominant slab membrane action developed in the presence of planar restraints. For the elevated temperature situation, an initial deflection of approximately 600mm is predicted by ADAPTIC, but it seems that at this stage the slab is already near failure, and hardly any temperature loading can be further sustained. The initial deflection obtained by SAFIR is relatively larger (about 800mm). A further deflection is developed at the beginning of the heating process, but a quite early numerical failure occurs at 170°C.

![Graph showing load-deflection response](image)

Fig. 3.53 Ambient response of slabs with and without planar restraint
In view of this, it can be concluded that although a compressive ring can be developed in slabs without planar restraint which would sustain tensile membrane action, the provision of planar restraint can enhance tensile membrane action and increase the slab resistance significantly.

Effect of secondary beams

Figures 3.55 and 3.56 provide the load-deflection response at ambient temperature and the deflection-temperature response at elevated temperature of the slab, where the reference slab is compared with the slab including the additional secondary beams. Under ambient conditions, the benefit of introducing the secondary beams is evident, as expected, due to the composite action of the floor system. On the other hand, for the elevated temperature response predicted by ADAPTIC, adding the secondary beams is effective in enhancing the load resistance only below a temperature of approximately 230°C. After this temperature, the deflection of the slab with the secondary beams starts to converge with and finally exceeds the deflection of the slab without secondary beams. This phenomenon implies that during the fire, the secondary beams under a significant compressive force due to the thermal expansion effects may exert a negative effect on the load carrying capacity of the slab. This is because the compressive force induced by thermal expansion and arching effects can introduce a larger deflection of the beam, thus tending to ‘pull down’ the whole slab system. Similar responses are predicted by SAFIR, where the deflection of the slab with the secondary beams is slightly smaller.
than the ADAPTIC prediction. This is due to a slight difference of the beam modelling strategy in SAFIR, where the geometric central axis of the secondary beams is considered in the same horizontal line as that of the slab, thus less compressive arching effect is developed, leading to a less significant ‘pulling down’ effect.

![Graph](image1)

**Fig. 3.55** Ambient response of slabs with and without secondary beams

![Graph](image2)

**Fig. 3.56** Elevated temperature response of slabs with and without secondary beams

### 3.6 Concluding remarks

This chapter discusses the ambient and elevated temperature responses of individual structural members including steel/composite beams, steel columns and slabs. An
important aim of this study has been to verify the capability of ADAPTIC models in simulating the behaviour of individual members under fire with sufficient accuracy, which is an important step towards confidently establishing more complex system models in further studies. With this aim, emphasis is given to the behaviour of the ADAPTIC models which are compared with available test data and numerical predictions obtained by other researchers, and favourable correlations are found. Apart from the benchmark studies, further discussions are presented to investigate the influence of various parameters on the behaviour of individual structural components. The corresponding comments and outcomes are noted as follows.

Boundary restraint conditions and thermal expansion effects have a significant influence on the behaviour of steel beams under fire. Normally, axial restraints with higher rigidity can lead to increased deflections of the steel beam at the beginning of fire. However, as the deflection increases, the provision of axial restraints can have the opposite effect of reducing the beam deflection due to the formation of catenary action. Regarding the temperature distribution over the depth of the steel beam, quadratic (parabolic) interpretations are usually sufficient. Idealised linear or uniform temperature distribution can yield a fair prediction; hence they can also be used in preliminary assessments when real temperature distributions are not immediately available. For composite beams, the 1D beam-column elements with rigid links in ADAPTIC has sufficient accuracy in simulating the behaviour of the concrete slab flange that is connected to the steel beam with rigid shear studs. Of course, for a detailed model of a floor system featuring membrane effects, shell elements would still be required in order to have a more reliable modelling of the floor slab.

Thermal expansion effects have a negligible influence on the fire behaviour of isolated columns without axial restraint. However, when a column belonging to a structural system is considered, the existence of axial restraints reduces the column buckling temperature, but may subsequently offer additional resistance during a post-buckling stage, provided that sufficient ductility/resistance is provided by the restraining structure. The provision of rotational restraints is also found to increase the column resistance, regardless of axial restraint conditions.

The shell elements implemented in ADAPTIC are verified to predict the ambient and elevated temperature responses of composite slabs very well. The ribbed slab model and
the flat slab model with an equivalent uniform thickness give similar results for the considered geometric configuration of the slab. This suggests that it is possible to adopt an equivalent uniform-thickness slab to simulate a ribbed slab in order to reduce modelling complexity while maintaining sufficient accuracy. However, this finding is based on the fact that the temperature distribution over the slab depth is assumed to be linear in this study. When a more real nonlinear temperature distribution is considered, a more significant difference between the ribbed slab and the corresponding simplified flat slab may be found under fire. Moreover, planar restraints have considerable influence on the slab behaviour. The resistance of the slab without planar restraint is much lower compared with the same slab with planar restraint. Finally, the influence of the secondary beams is investigated, where their relative contribution at high temperatures is found to be greatly reduced with the loss of their strength and stiffness. Therefore, the steel beam (or part of the steel beam) subject to high temperatures may be seen as ‘removed’. This idea forms the basis of the Temperature-Independent Approach (TIA) introduced in Chapter 8. In addition, the combined effects of thermal expansion and thermal bowing may further increase the vertical deflection of the beam. Consequently, the deflection of the floor system with secondary beams under high temperatures can be similar to or even slightly larger than the floor response without secondary beams. This phenomenon is, however, only based on the assumption that a fully developed fire leads to a high temperature along the whole length of the steel beam. Under a localised fire scenario where the high temperature area is limited, the adjacent part of the beam exposed to fire can be less affected, so the ‘pulling down’ effect would be reduced.

Considering the above, the responses of individual members predicted by ADAPTIC compare generally well with the selected test and numerical results. In addition, the outcomes from the undertaken studies are consistent with related results found in other studies reviewed in Chapter 2, which further verifies the reliability of the ADAPTIC models. In the next chapter, the response of joints under ambient and fire conditions is addressed in detail.
CHAPTER 4

Joint Response and Ductility Supply

4.1 Introduction

It has been acknowledged that joints play an important role in providing ductility for steel-composite structures subject to extreme loading conditions, such as blast, fire and impact. Under these loading conditions, extensive inelastic deformations are expected, which are usually associated with excessive ductility demands concentrated in the joint regions near the affected structural members in typical steel-composite structures. Moreover, the resistance of joints exposed to lasting fires can significantly deteriorate. Therefore, the ability of these joints to accommodate excessive ductility demands without failure at elevated temperature becomes an essential factor for the mitigation of progressive collapse. The aim of this chapter is to discuss the modelling techniques of joints under ambient and elevated temperatures, and subsequently to define a joint failure criterion which can be incorporated into the robustness assessment framework for further studies.

The behaviour of joints under fire has been extensively investigated. Traditional anti-fire treatments for joints consist of applying a fire protection coating or concrete encasement, where the joints can be designed for reduced or even ambient temperature. However, applying joint fire protection is costly, so over the last decade there has been more interest in understanding the behaviour of unprotected joints. Extensive experiments have been undertaken to study the unprotected joint response, which were comprehensively summarised by Al-Jabri et al. (2008). In addition, analytical approaches are also widely accepted to predict the joint response with reasonable accuracy. Commonly used analytical approaches include curve-fitting expressions of
previous experimental results, detailed 3D finite element models, and component methods. Among the various available assessment methods, tests on joints are the most realistic, but experimental data is only available for limited types of joints, and there are even discrepancies in the response between joints of the same type. Curve-fitting expressions can provide an efficient way for predicting the moment-rotation response of joints, and it can be easily included in frame models, but its application is limited to the specific joints that have been tested. Detailed finite element (FE) modelling offers an alternative reliable approach that can accurately predict the full response of joints; however, 3D joint FE models are computationally expensive and are difficult to incorporate directly in a global frame model due to their complexity.

The component method or the spring-stiffness method was first introduced in Eurocode 3 (2005a) as a simplified analytical approach for predicting the joint behaviour under ambient conditions. The principle of this method is to subdivide a joint into several tension zones (T-stubs) and a compression zone, and each zone is further divided into basic components of known characteristics. The component method can possess sufficient accuracy if the properties (i.e. force-deformation relationship) of each component are represented accurately. In current design practice, the application of this method is mainly focused on obtaining the moment-rotation relationship of major axis end-plate joints under ambient conditions, and applying this relationship for rotational springs embedded at the position of the joints in a global frame model (Figure 4.1a). In this frame model, the influence of the axial force on the joint response is not considered, which is unrealistic under localised fire scenarios, where the column can be severely damaged and a large deflection of the floor system can consequently develop. To address this, a set of axial springs with predefined stiffness and resistance can be alternatively employed into the global frame model to represent individual bolts rows and compression zones, as illustrated in Figure 4.1b. In case of fire, temperature effects can be considered by degrading the properties of these springs. Through this, the axial force and moment resistances of the joints are coupled. Furthermore, the spring approach is capable of simulating uneven temperature distributions across the joint, which can be implemented by degrading each spring independently based on the different temperatures at the locations of each bolt-row. In view of the advantages stated above, the component method (Figure 4.1b) is considered for joint modelling in the current study due to its reliability, accuracy and convenience.
In Section 4.2, general design procedures of the component method specified in Eurocode 3 (2005a) are briefly introduced, where the properties of various joint components are presented in detail. Subsequently, joint failure criteria are defined in Section 4.3, and this is followed in Section 4.4 by a discussion on the influence of elevated temperature on the joint response. Finally, available joint tests conducted by other researchers are used in Section 4.5 to compare with the component joint models considered in this study, where focus is given to the comparison of moment-rotation relationships and ductility supplies. The influence of the post-limit stiffness of ductile components on elasto-plastic joint response is also studied.

### 4.2 General design procedure of component method

Two steps are generally involved in the codified component method in order to establish the joint moment-rotation response. Considering a typical steel joint as shown in Figure 4.2a, the first step is to decompose the joint into several bolt-row/bolt-row group springs and contact springs as illustrated in Figure 4.2b, which can be further decomposed into a series of individual active components. The second step is to assemble the properties of these components and incorporate them into a rotational spring featuring the obtained moment-rotation characteristic, as shown in Figures 4.2c and 4.2d. The moment resistance and stiffness of the joint can be derived from the resistances and stiffness of its basic components to the applied rotation. The obtained
moment-rotation relationship can be employed in further elastic-plastic global analysis on a frame model. The general procedures specified in Eurocode 3 (2005a) for the two main steps are introduced briefly next.

Fig. 4.2 Joint moment-rotation characteristic: a) joint configuration, b) spring assembly, c) rotational spring model, d) moment-rotation curve

4.2.1 Step 1 – component response

The moment-rotation response is determined by the force-deformation characteristics of the components. Most of the strength and stiffness expressions are calculated based on the underlying mechanics of idealised components, which are normally obtained with evidence from available sophisticated numerical and experimental studies. Figure 4.3 illustrates a typical flush end-plate steel joint which can be decomposed into the individual components as noted. Each component normally has two important elastic
characteristics, which are the design resistance $F_{Rd}$ and the stiffness coefficient $k$, where $k$ is a general coefficient and the real stiffness is obtained by multiplying $k$ with $E$ (elastic modulus of the material, e.g. steel). One of the novel features of the component method is to model the bending deformations of the end-plate, the column flange, and the steel bolts by means of an equivalent T-stub with an effective width, as illustrated in Figure 4.4a. Three possible failure modes are identified for the T-stubs, as shown in Figure 4.4b. Mode 1 is associated with a more flexible T-stub flange connected to relatively strong bolt-rows, and failure is governed by the yielding of the plate. When a stronger T-stub flange is employed, mode 2 occurs where both plate and bolts govern the failure mode. Mode 3 corresponds to a brittle failure type with much thicker plates, where the rupture of bolts occurs prematurely. The third failure mode should be avoided in joint design if a sufficient ductility supply of the joint is required.

Although the current application of the component method in design codes refers to specific joint configurations with respect to specific axis, e.g. end-plate joints with respect to major axis, the component method can in principle be applied to practically any type of joint with respect to either major or minor axis by decomposing the joint into the relevant components, provided that each component is comprehensively investigated. Due to different roles that components play in a certain equilibrium state, three component zones can be identified, namely, tension zone components, compressive zone components, and shear zone components.

![Fig. 4.3 Typical joint components (Vlassis, 2007)]
Table 4.1 lists the components that are normally considered in several widely used steel bolted joint types (single-sided) under hogging moments, where the components considered for each joint type can be selected from a ‘component library’. For the web/flange cleat joint and the fin plate joint in the current table, the bottom beam flange is assumed not to be in direct contact with the column face in the compressive zone due to the existence of gaps. However, when a more considerable rotation is considered, additional compressive zone components, e.g. beam web/flange in compression, should be added to represent the gap closure. Moreover, when double-sided joints are considered, some components given in Table 4.1 may be deactivated, e.g. column web in shear under equal and opposite moments.

The design resistance $F_{Rd}$ and stiffness coefficient $k$ expressions for all the components in end-plate joints and top/bottom flange cleat joints are specified in detail in Eurocode 3 (2005a). For web cleat joints which are not directly discussed in Eurocode3 (2005a), an equivalent T-stub approach can be employed for the component ‘web cleat in bending’, which is similar to that adopted for end-plate joints. Regarding minor axis joints, most of the adopted components are similar to those considered in major axis joints, except the component ‘column web in bending’, as shown in Figure 4.5.
Table 4.1 Active components of single-sided joints

<table>
<thead>
<tr>
<th>Component library:</th>
<th>Active components</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) column web panel in shear; (2) column web in transverse compression; (3) column web in transverse tension; (4) column flange in bending; (5) column web in bending; (6) end-plate in bending; (7) angle/cleat in bending; (8) angle/cleat/fin plate in tension; (9) angle/cleat/fin plate in compression; (10) angle/cleat in bearing; (11) beam flange/web in compression; (12) beam web in tension; (13) bolts in tension; (14) bolts in shear; (15) bolts in bearing.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bending axis</th>
<th>Zones</th>
<th>Flush end-plate joint</th>
<th>Top and bottom seat cleat joint</th>
<th>Web cleat joint</th>
<th>Fin plate joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment applied with respect to column major axis</td>
<td>Tension zone</td>
<td>(3), (4), (6), (12), (13)</td>
<td>(3), (4), (7), (8), (10), (13), (14), (15)</td>
<td>(3), (4), (7), (8), (10), (12), (13), (14), (15)</td>
<td>(3), (8), (10), (12), (14), (15)</td>
</tr>
<tr>
<td>Compressive zone</td>
<td>(2), (11)</td>
<td>(2), (9), (10), (14), (15)</td>
<td>(2), (9), (10), (14), (15)</td>
<td>(2), (9), (10), (14), (15)</td>
<td></td>
</tr>
<tr>
<td>Shear zone</td>
<td>(1)</td>
<td>(1)</td>
<td>(1)</td>
<td>(1)</td>
<td></td>
</tr>
</tbody>
</table>

| Bending moment applied with respect to column minor axis | Tension zone | (5), (6), (12), (13) | (5), (7), (8), (10), (13), (14), (15) | (5), (7), (8), (10), (12), (13), (14), (15) | (5), (8), (10), (12), (14), (15) |
| Compressive zone | (2), (5), (11) | (5), (9), (10), (14), (15) | (5), (9), (10), (14), (15) | (5), (9), (10), (14), (15) |
| Shear zone | Not applicable | Not applicable | Not applicable | Not applicable |

Lima et al. (2002) proposed analytical expressions for obtaining the stiffness coefficient $k$ of the component ‘column web in bending’, which is given by:

$$ k_{cw} = 16 \frac{t_{wc}^3}{L^2} \left( \frac{\alpha + (1-\beta)\tan \theta}{(1-\beta)^3} + 10.4 (k_1 - k_2 \beta) \right)/\mu^2 $$

(4.1)

where $\alpha = c/L$, $\beta = b/L$, $\mu = L/t_{wc}$, $k_1 = 1.5$, $k_2 = 1.63$, $\theta = 35 - 10\beta$, and other symbols are given in Figure 4.5.
Considering the influence of adjacent column web stiffeners, Eq. (4.1) can be modified with a polynomial approximation, as given by:

\[
k_{cw} = 16 \frac{t_w c}{L^2} \left[ \alpha + (1 - \beta) \tan \theta \right] + 10.4 \left( k_i - k_2 \beta \right) / \mu^2 \left[ 0.57 \left( \frac{m}{D} \right)^2 - 0.85 \left( \frac{m}{D} \right) + 0.41 \right]
\]

where \( m \) is the distance between the top stiffener and the load application point, \( D \) is the distance between the stiffeners, as shown in Figure 4.5. Due to the assumption of a rectangular loading area, the proposed expressions are more accurate for column web in bending in the compression zone.

\[
k_{cw} = \frac{\pi t_c^3}{12 \left( 1 - \nu^2 \right) C_i \left( b_c - t_c \right)^2}
\]

where \( t_c \) is the column face thickness, \( \nu \) is the Poisson’s ratio for steel, \( b_c \) is column face width, \( C_i \) is a coefficient normally taken as 0.18.

**Fig. 4.5** Column web in bending in minor axis joint (Lima et al. 2002)

Málaga-Chuquitaype and Elghazouli (2010) also proposed equations for calculating the design resistance \( F_{rd} \) and stiffness coefficient \( k \) of the component ‘column web in bending’, where focus was initially given to square tubular column face in bending. These equations are more suitable for column web in bending in the tension zone. The initial stiffness coefficient is:
The design resistance $F_{Rd}$ is considered as the smaller value of the punching resistance ($F_{Rd1}$) and the local yielding capacity ($F_{Rd2}$), where $F_{Rd1}$ and $F_{Rd2}$ are expressed by:

$$F_{Rd1} = \frac{N \pi D_h t_c f_y}{\sqrt{3}}$$, if $2(L + 0.9D_h) \geq N \pi D_h$  \hspace{1cm} (4.4)

$$F_{Rd1} = \frac{2(L + 0.9D_h)t_c f_y}{\sqrt{3}}$$, if $2(L + 0.9D_h) < N \pi D_h$  \hspace{1cm} (4.5)

$$F_{Rd2} = f_k \gamma m_{plc}$$  \hspace{1cm} (4.6)

where $m_{plc}$ is the plastic bending moment of the column face per unit length, $D_h$ is the diameter of the bolt hole, $N$ is the number of bolts, $f_y$ is the yield stress, $L$ is the spacing between the two bolts in the same bolt-row, and factors $f_k$ and $\gamma$ are expressed by:

$$f_k = 1$$, if $\frac{L + 0.9D_h}{b_c} \geq 0.5$  \hspace{1cm} (4.7)

$$f_k = 0.7 + \frac{0.6(L + 0.9D_h)}{b_c}$$, if $\frac{L + 0.9D_h}{b_c} < 0.5$  \hspace{1cm} (4.8)

$$\gamma = \frac{4}{1 - L/b_c} \left( \pi \sqrt{1 - \frac{L}{b_c}} \right) + 1.8 \frac{D_h}{b_c}$$  \hspace{1cm} (4.9)

Finally, when a composite joint under hogging moments is considered, Eurocode 4 (2004) recommends that the reinforcing bars in tension can be treated similarly to the other components specified in Eurocode 3 (2005a). In this respect, the component method employed for predicting the behaviour of steel joints can be extended to composite joints with an additional component representing the reinforced concrete flange. It is suggested that the characteristic of the component of rebars should be based on tests or on analytical or numerical methods supported by tests. Calculation methods may be also used provided that they are supported by tests. For a double-sided composite joint, the rebars have to be continuous overhead to develop composite action. For a single-sided configuration designed as a composite joint, the rebars in tension
should be anchored sufficiently well beyond the span of the beam to enable the design tension resistance to be developed.

4.2.2 Step 2 – joint moment-rotation response

The design moment resistance $M_{j,Rd}$ and the rotational stiffness $S_j$ are two important parameters determining the moment-rotation characteristic of joints. In elastic global analysis, a linear relationship between the bending moment and the joint rotation is assumed, as illustrated in Figure 4.6a-b. If the design bending moment $M_{j,Ed}$ does not exceed $2/3 M_{j,Rd}$, the initial rotational stiffness $S_{j,ini}$ may be taken directly in the global analysis. As a simplified and conservative approach, the rotational stiffness $S_j$ may be taken as $S_{j,ini} / \mu$ in the analysis for all values of the design bending moment $M_{j,Ed}$, where $\mu$ is the stiffness modification coefficient specified in Eurocode 3 (2005a). When an elasto-plastic global analysis is conducted, the post-yield response can be considered in an idealised bilinear manner, as shown in Figure 4.6c.

\[ M_{j,Rd} \]

\[ \sum_r h_r F_{r,Rd} \]

where $F_{r,Rd}$ is the effective design tension resistance of bolt-row $r$, and is taken as the smallest component resistance in that bolt-row; $h_r$ is the distance from bolt-row $r$ to the centre of compression, for bolted end-plate connections, the centre of compression should be assumed to be in line with the centre of the compression flange of the
connected member, provided its compressive resistance exceeds the tensile resistance, and \( r \) is the bolt-row number starting from the furthest bolt-row from the centre of compression. The design tension resistance \( F_{tr,Rd} \) for each bolt-row should be determined in sequence from bolt-row 1 (furthest from the centre of compression) towards nearer bolt-rows. For each bolt-row considered, the resistance \( F_{tr,Rd} \) should be calculated either individually or as a group with the bolt-rows further away from the centre of compression, whichever is conservative. All other bolt-rows closer to the centre of compression should be ignored.

**Determination of rotational stiffness \( S_j \)**

Provided that the axial force applied on the connected member (e.g. beam) does not exceed 5% of the design resistance \( N_{pl,Rd} \) of its cross-section, the rotational stiffness \( S_j \) of a typical major axis end-plate joint subject to a bending moment of \( M_{j,Ed} \) less than the design moment resistance \( M_{j,Rd} \) can be calculated from:

\[
S_j = \frac{E_s z_{eq}^2}{\mu \left( \frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_{eq}} \right)}
\]  

(4.11)

where \( E_s \) is the elastic modulus of steel, \( k_i \) is the stiffness coefficient of shear zone components, \( k_2 \) is the stiffness coefficient of compression zone components, \( k_{eq} \) is a single equivalent stiffness representing the characteristic of all bolt-rows, as given by:

\[
k_{eq} = \frac{\sum k_{eff,r} h_r}{z_{eq}}
\]  

(4.12)

where \( k_{eff,r} \) is the effective stiffness coefficient for each bolt-row calculated from all the active components in this bolt-row, \( z_{eq} \) is the equivalent lever arm given by:

\[
z_{eq} = \frac{\sum k_{eff,r} h_r^2}{\sum k_{eff,r} h_r}
\]  

(4.13)
With respect to the maximum rotation capacity of bolted joints, Eurocode 3 (2005a) briefly presents several specific conditions where an adequate rotation capacity can be assumed for the joint in plastic global analysis, otherwise tests are recommended. While only a brief statement on joint rotation capacity is provided in the code, the next section discusses the ductility supply and failure criteria of joints in more detail.

### 4.3 Joint failure criteria

Unlike normal loading conditions, larger deflections are inevitable when structures are subjected to extreme loading. The vulnerability to progressive collapse of a structure greatly depends on the capability of the joints to withstand large deformations. A favourable joint ductility is important to ensure that a sufficient deformation capacity is available, thus preventing joint failure and progressive collapse. Predictions of joint ductility supply require a clear definition of failure criteria that can explicitly account for the ductility supply of each individual joint component. In other words, the joint ductility capacity is closely associated with the deformation capacity of each active component. Although relevant research has been started on this front (Spyrou et al., 2004a, 2004b), available information with regard to the joint component ductility supply under fire conditions is still mainly empirical and insufficient.

![Fig. 4.7 Load-deformation relationships for components](image)

The behaviour of each component can be represented by a force-deformation curve, which is usually characterised by bilinear, trilinear or general nonlinear relationships, as illustrated in Figure 4.7. For the bilinear curves, the first segment represents the elastic performance with a stiffness of $k_e$ before yielding, which is followed by a post-limit curve with a stiffness $k_{pl} = \mu_1 k_e$, where $\mu_1$ is the strain hardening coefficient. When a further reduced stiffness $\mu_2 k_e$ needs to be considered beyond the ultimate strength $f_{tu}$, a tri-linear curve can be adopted. While general nonlinear curves may represent the
component force-deformation response more accurately, their definition is more complex than the piecewise linear approximations.

A piecewise linear force-deformation curve (e.g. bilinear curve) for each component is comprised of four key parameters, namely, elastic resistance \( F_{Rd} \), elastic stiffness \( k \), post-limit (post-elastic) stiffness \( \mu_k \), and maximum deformation capacity \( u_{\text{max}} \). The calculations of the first two parameters (elastic resistance and elastic stiffness) for the joint components have already been comprehensively presented in Eurocode 3 (2005a), as discussed in Section 4.2, but so far fewer investigations have been devoted to a systematic evaluation of the post-limit response (post-limit stiffness and maximum deformation capacity) for each individual component. Therefore, the focus of this section is to discuss the post-limit stiffness and the maximum deformation capacity of each component.

### 4.3.1 Component post-limit stiffness

The component post-limit stiffness is typically obtained through multiplying the elastic stiffness with a strain hardening coefficient. Eurocode 3 (2005a) suggests that the post-limit stiffness can be conservatively taken as zero in elasto-plastic global analysis. In reality, however, the post-limit stiffness may be considerable for some components, especially for highly ductile components; hence, the neglect of the post-limit stiffness may lead to an underestimation of the bending resistance of joints. In recent studies focusing on the post-limit stiffness of components, bilinear and trilinear component force-deformation curves were mainly used and studied, where the main findings are introduced as follows.

Employing the bilinear component force-deformation response, Ren and Crisinel (1995) adopted the value of 6% for ductile components in double web-cleat joints. Simões da Silva et al. (2002) proposed various strain hardening coefficients for different ductile components in typical joint types based on available test results. Bilinear curves were employed to simulate the load-deformation characteristics for each component. The post-limit stiffness of four typical components was predicted, which are column web panel in shear, column web in compression, column web in tension, and column flange in bending. An iteration procedure was adopted to make the closest comparison with the experimental data chosen from the database of steel joints SERICON II (Cruz et al. 1998), through which the post-limit stiffness for each component was predicted. Based
on the results, general values of the post-limit stiffness for the four components were suggested, where the values generally range from 1% to 5%. Lima et al. (2005) proposed a similar iteration approach called the ‘Genetic Algorithm Approach’ to obtain the post-limit stiffness of eight commonly used components using bilinear curves. Five sets of experimental data were used for calibration. The first one was a flush end plate joint tested at the Civil Engineering Department of the University of Coimbra. The other four beam-to-column joint tests were chosen from SERICON II database (two extended endplate joints with backing plates and two welded joints). Although final recommendations for the post-limit stiffness value of the eight components were not systematically concluded, the ranges of these for the five experiments were provided.

Considering the trilinear approach, Al-Jabri et al. (2004) employed the strain hardening coefficients $\mu_1$ and $\mu_2$ of 0.05 and 0.01 respectively to predict the post-limit component force-deformation relationship. These values were also adopted by Ramli-Sulong et al. (2007), who developed a component-based joint model in the nonlinear FE program ADAPTIC (Izzuddin, 1991). These models were shown to be capable of predicting the moment-rotation relationships of joints with favourable accuracy. Savio et al. (2009) proposed a generalised component-based model for beam-to-column connections, and the moment-axial force interaction was considered. The trilinear approach was adopted for the load-deformation relationship for all the considered components, and the corresponding values of the strain hardening coefficients $\mu_1$ and $\mu_2$ were provided.

Table 4.2 summarises the values of the component post-limit stiffness studied by various researchers as discussed above. It can be seen that the predictions obtained from different researchers can vary considerably, which may be due to different joint configurations and material properties considered, as well as the diverse modelling methods. Despite the high level of uncertainty, it is found that most predictions of the component post-limit stiffness fall between 1% and 5% for the bilinear response, except the predictions from Savio et al. (2009), who gave much higher values of strain hardening coefficients. The influence of different post-limit stiffness of joint components on the overall joint behaviour will be further investigated in the last section of this chapter, where responses using the ADAPTIC multi-spring joint models assuming different post-limit characteristics are compared with available test data.
### Table 4.2 Summary of component post-limit stiffness

<table>
<thead>
<tr>
<th>Components</th>
<th>Bi-linear approach</th>
<th>Tri-linear approach</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Simões da Silva et al. (2002) $\mu_1$</td>
<td>Lima et al. (2005) $\mu_1$</td>
</tr>
<tr>
<td>Column web in shear</td>
<td>4.6%</td>
<td>1.76-7%</td>
</tr>
<tr>
<td>Column web in compression</td>
<td>2.3%</td>
<td>3.07-9.28%</td>
</tr>
<tr>
<td>Column web in tension</td>
<td>0.1-1.7%</td>
<td>0.25-15%</td>
</tr>
<tr>
<td>Column flange in bending</td>
<td>1.3%</td>
<td>0.13-15%</td>
</tr>
<tr>
<td>Endplate in bending</td>
<td>–</td>
<td>0.18-1.84%</td>
</tr>
<tr>
<td>Beam web in tension</td>
<td>–</td>
<td>2.05-3.65%</td>
</tr>
<tr>
<td>Beam flange/web in compression</td>
<td>–</td>
<td>0.44-4.89%</td>
</tr>
<tr>
<td>Bolt in tension</td>
<td>–</td>
<td>0.49-8%</td>
</tr>
</tbody>
</table>

### 4.3.2 Component deformation capacity

Apart from the component post-limit stiffness that contributes to the joint post-elastic behaviour, the ultimate component deformation is another significant post-limit parameter used for predicting joint failure. A component classification method with respect to ductility was firstly proposed by Kuhlmann et al. (1998), where three main ductility classes were identified, namely, high ductility joint components, limited ductility joint components, and brittle joint components. For the components falling into the high ductility category, the force-deformation curve transfers from the linear elastic range into a second stage exhibiting increased resistance with deformation. The deformation capacity of the high ductility component was typically defined as infinite. The components that belong to the limited ductility class are characterised by the load-deformation curves with an elastic range and a subsequent limited post-limit response (either hardening or softening) followed by failure. For brittle components, little post-yield deformation is achieved, so failure is conservatively assumed at the point where
yield strength is attained. The force-deformation responses (trilinear approximation) of the three component classes are illustrated in Figure 4.8.

![Diagram showing the force-deformation responses for high, limited, and brittle ductility classes.](image)

**Fig. 4.8** Piecewise linear approximation of three component ductility classes

Based on this concept, a ductility index $\phi = d_f/d_y$ for each component was further developed by Kuhlmann *et al.* (1998) and Simões da Silva *et al.* (2002), where $d_f$ is the maximum deformation, and $d_y$ is the elastic yielding deformation for each component. For the components with high ductility, the ductility index $\phi$ is suggested as infinite or at least larger than 20. For limited ductility class components, a ductility index value of 3 to 20 is recommended. For brittle components, little post yield deformation is performed, so the ductility index $\phi$ can be conservatively taken as 1.0. The elastic response and ductility index for typical joint components considered for end-plate joints and web cleat joints are provided in Table 4.3, where most of the listed ductility indices are based on Silva *et al.* (2002) except the component ‘plates in bearing’ which was not initially provided. Although this component does not need to be considered in end-plate joints, it is important for cleat joints and fin plate joints. Generally for a typical double web-cleat angle joint, plate bearing components include the angle in bearing and the beam web in bearing. Sarraj (2007) investigated various factors that affect the plate bearing behaviour through an intensive finite element parametric study. An effective nonlinear curve fitting approach was developed to predict the load-deformation response of the ‘plate in bearing’ component under both ambient and elevated temperature conditions. This curve fitting approach was an extension of the research undertaken by Rex and Easterling (2003) for the load-deformation response of the ‘single bolt single plate’ situation under room temperature.
Table 4.3 Elastic response and ductility classification for each component

<table>
<thead>
<tr>
<th>Ductility index</th>
<th>Component</th>
<th>Elastic Resistance</th>
<th>Elastic Stiffness</th>
<th>Limit deformation ($d/d_0$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column web in shear</td>
<td>$F_{R,wc} = 0.9 f_{y,wc} A_{wc}$</td>
<td>$K_{wc} = E \frac{0.38 A_{wc}}{\beta z}$</td>
<td>Infinite</td>
</tr>
<tr>
<td></td>
<td>Column web in tension</td>
<td>$F_{R,wc} = 0.7 b_{eff,wc} f_{y,wc}$</td>
<td>$K_{wc} = E \frac{0.7 b_{eff,wc} t_{wc}}{d_{y}}$</td>
<td>Infinite</td>
</tr>
<tr>
<td></td>
<td>End-plate in bending</td>
<td>Equivalent T-stub model</td>
<td>$K_{sbh} = E \frac{0.85 l_{eff} f_{te}}{m^3}$</td>
<td>Infinite</td>
</tr>
<tr>
<td></td>
<td>Column flange in bending</td>
<td>Equivalent T-stub model</td>
<td>$K_{sbh} = E \frac{0.85 l_{eff} f_{te}}{m^3}$</td>
<td>Infinite</td>
</tr>
<tr>
<td></td>
<td>Angle in bending</td>
<td>Equivalent T-stub model</td>
<td>$K_{ab} = E \frac{0.85 l_{eff} f_{te}}{m^3}$</td>
<td>Infinite</td>
</tr>
<tr>
<td></td>
<td>Angle plate in bearing</td>
<td>$F_{R,ab} = k_a f_{u} d_1 t_{u}$</td>
<td>$K_{ab} = 24 n_k k_d f_{u}$</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Beam web plate in bearing</td>
<td>$F_{R,bwc} = k_a f_{u} d_1 t_{w}$</td>
<td>$K_{bwc} = 24 n_k k_d f_{u}$</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>Column web in compression</td>
<td>$F_{R,cwc} = 0.5 k_h f_{u} d_1 t_{wc}$</td>
<td>$K_{cwc} = E \frac{0.7 b_{eff,wc} f_{y,wc}}{d_{y}}$</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Bolts in tension</td>
<td>$F_{R,bl} = 2 k_f f_{u} A_{f}$</td>
<td>$K_{bl} = 1.6 E A_f / L_{f}$</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Bolts in shear</td>
<td>$F_{R,bl} = n_f A_{f}$</td>
<td>$K_{bl} = \frac{16 n_f d_{f}^2 f_{u}}{d_{bl}^2}$</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Beam flange/web in compression</td>
<td>$F_{R,bwc} = \frac{M_{bwc}}{z}$</td>
<td>$\infty$</td>
<td>1</td>
</tr>
</tbody>
</table>

Based on Sarraj (2007), the load-deformation relationship of the component ‘plate in bearing’ for a small end distance bearing case ($e_2 \leq 2d_b$) is given by:

$$\frac{F}{R_{b,rd}} = \frac{\Psi \Delta}{\left(1 + \Delta^{0.5}\right)^{2}} - \Phi \Delta$$  \hspace{1cm} (4.14)

where $F$ is the applied load, $R_{b,rd}$ is the resistance for plate bearing $R_{b,rd} = \frac{e_2}{d_{b}} f_{u} d_{b} t$, $f_{u}$ is the ultimate strength of the plate, $e_2$ is the end distance in the direction of loading,
$d_b$ is the bolt diameter, $\Delta$ is the normalised deformation, $\Delta = \Delta \beta K_i / R_{b,i}$, $\Delta$ is the bolt-hole elongation, $\beta$ is usually taken as 1 for typical steel, $K_i$ is the initial stiffness including three parameters:

$$K_i = \frac{1}{K_{br} + \frac{1}{K_b} + \frac{1}{K_v}} \quad (4.15)$$

Bearing stiffness $K_{br} = \Omega t f_y (d_b / 25.4)^{0.8}$ \quad (4.16)

Bending stiffness $K_b = 32Et (e_2 / d_b - 0.5)^3$ \quad (4.17)

Shearing stiffness $K_v = 6.67Gt (e_2 / d_b - 0.5)$ \quad (4.18)

In the above, $f_y$ is the plate yield strength, while $\Omega$, $\Psi$ and $\Phi$ are temperature-dependent parameters, taking the values of 145, 2.1, and 0.012, respectively, under ambient temperature. The values of $\Omega$, $\Psi$ and $\Phi$ recommended under elevated temperature conditions are presented in Sarraj (2007).

![Fig. 4.9 Force-deformation response for the plate bearing component](image)

Eurocode 3 (2005a) also provides expressions for the resistance and initial stiffness of the plate bearing component under ambient conditions (as given in Table 4.3). Assuming a typical plate bearing case: $d_b = 20\text{mm}$, $e_2 = 2d_b$, $E = 200\text{Gpa}$, $f_y = 355\text{Mpa}$, $f_u = 455\text{Mpa}$, and $t = 10\text{mm}$, the load-deformation curves from the two methods are compared in Figure 4.9. The elastic response obtained from Sarraj’s method and EC3
method are similar, while the EC3 method ignores the post-limit stiffness if no strain hardening is considered. When a strain hardening coefficient of 3% for the post-limit stiffness is assumed in the EC3 method, a closer response to Sarraj’s method is found. It can be also observed that the maximum/ultimate resistance of the plate in bearing predicted by Sarraj’s method is approximately 10mm, leading to a component ductility index $d_u/d_y$ of around 15. According to the parametric study conducted by Sarraj (2007), the value of 15 is a general ductility index $d_u/d_y$ for most of the other cases. Therefore, in the current study, a ductility index $d_u/d_y$ of 15 is adopted for all the plate bearing components (classified as ‘limited ductility’ components).

Each component has now been categorized into its corresponding ductility classification, and the ductility index $d_u/d_y$ for all the components is determined. Notwithstanding, as noted in Table 4.3, all the ‘high ductility’ components have an infinite deformation capacity in tension, which is unrealistic. Vlassis (2007) suggested a 30mm for the tensile deformation capacity of each bolt-row in typical end-plate and web-cleat joints. This value was in accordance with relevant test data (Jarrett, 1990; Owens and Moore, 1992), where a large number of flexible end-plate and double-web angle connections were tested under tensile force until failure. According to Jarrett (1990), the average axial deformation capacities for the end-plate and web-cleat joints in the tests were 25.4mm and 37.2mm, respectively. From the test results of Owens and Moore (1992), the average axial deformation capacities for the same two types of joint were 26.8mm and 37.3mm, respectively. Therefore, in this study, the maximum deformations of 25mm and 35mm are employed as an additional failure criterion for all bolt-rows in end-plate connections and cleat/angle/fin plate connections, respectively.

When composite action is considered, the ductility of composite joints not only relies on the deformation capacity of the steel component discussed above, but is also associated with the rupture of reinforcement rebars. Due to steel-concrete bond action and cracking of concrete, the reinforcement stress is concentrated in the vicinity of the cracking, which leads to the rupture of the reinforcement bars only at crack locations. The CEB-FIP model code (1990) gives a simplified stress-mean strain relationship for embedded reinforcement and compares with the corresponding curve for bare reinforcement (as shown in Figure 4.10). Based on this model, Anderson et al. (2000) developed a simplified calculation method for predicting the rotational capacity of composite joints limited by ultimate reinforcement elongations. This method has been proved to have
sufficient accuracy through validations against experiments, and it has been employed by various researchers (Vlassis et al., 2008; Gil and Bayo, 2008; Fu et al., 2010). The basic principle of the method is to predict the ultimate mean strain of embedded reinforcement $\varepsilon_{smu}$ that is related to the properties of bare reinforcement and concrete, and then to obtain the maximum reinforcement elongation $\Delta u,s$ through multiplying $\varepsilon_{smu}$ by an effective length. The application of the method is briefly discussed next.

Fig. 4.10 Simplified stress-strain relationship of embedded reinforcement (CEB, 1990)

The ultimate mean strain $\varepsilon_{smu}$ is calculated as follows:

$$
\varepsilon_{smu} = \varepsilon_{sy} - \beta_r \Delta \varepsilon_{sr} + \delta \left(1 - \frac{\sigma_{sr1}}{f_{yk}}\right) (\varepsilon_{su} - \varepsilon_{sy})
$$

(4.19)

where

- $\beta_r$ is taken as 0.4 for short-term loading,
- $\Delta \varepsilon_{sr}$ is the increase of reinforcement strain when the first crack forms, $\Delta \varepsilon_{sr} = \frac{f_{cm} k_c}{E, \rho}$,
- $\delta$ is taken as 0.8 for high ductility bars,
- $\sigma_{sr1}$ is the reinforcement stress at the location of first crack, $\sigma_{sr1} = \frac{f_{cm} k_c}{\rho} \left(1 + \rho \frac{E, c}{E, v}\right)$,
- $\rho$ is the longitudinal reinforcement ratio,
- $k_c$ is a coefficient that allows for the self-equilibrating stresses and the stress distribution in the slab prior to cracking, $k_c = \frac{1}{1 + d/2z_0}$,
- $d$ is the thickness of the concrete flange, excluding any ribs,
- $z_0$ is the vertical distance between the centroid of the uncracked, unreinforced concrete flange and the uncracked, unreinforced composite section, calculating using the modular ratio for short-term effects.
Finally, the ultimate reinforcement elongation $\Delta_{u,s}$ is obtained as:

$$\Delta_{u,s} = 2L_a, \varepsilon_{mu} \quad \text{for} \quad \rho < 0.8\%$$  \hspace{1cm} (4.20)

$$\Delta_{u,s} = \left(\frac{h_c}{2} + L_t\right), \varepsilon_{mu} \quad \text{for} \quad \rho \geq 0.8\% , \quad \text{and} \quad a < L_t$$  \hspace{1cm} (4.21)

$$\Delta_{u,s} = \left(\frac{h_c}{2} + L_t\right), \varepsilon_{mu} + (a - L_t)\varepsilon_{cmy} \quad \text{for} \quad \rho \geq 0.8\% , \quad \text{and} \quad a > L_t.$$  \hspace{1cm} (4.22)

where

- $a$ is the distance from the face of column to the first shear stud,
- $h_c$ is the column section depth,
- $L_t$ is the transmission length $L_t = \frac{k_c f_{cmt} \phi}{4\tau_{sm} \rho}$,
- $\phi$ is the diameter of the reinforcement bar,
- $\tau_{sm}$ is the average bond stress along the transmission length $\tau_{sm} = 1.8 f_{cmt}$.

With respect to the definition of shear failure criterion, shear action is considered in this study separately and uncoupled with bending and axial actions. Shear failure of joints is considered through incorporating a rigid-plastic load-deformation curve into the vertical shear property of the bottom spring of the joint model. The load-deformation response of the shear spring is illustrated in Figure 4.11.

![Rigid-plastic shear response of connections](image)

**Fig. 4.11** Rigid-plastic shear response of connections

Indeed, shear failure is less likely to occur in ambient joints, since they are usually designed with sufficient resistance reserve to consider the increased shear force due to column loss. Under fire conditions, however, the shear resistance of fire affected joints can be greatly impaired under high temperatures; therefore, the shear failure (punching
shear) of the fire affected floor has to be considered as a possible failure mode. Considering a floor system in fire as shown in Figure 4.12, shear failure is deemed to occur when the overall shear force applied onto the four steel beam-to-column connections exceeds their overall shear resistance at elevated temperatures. This failure criterion assumes a rigid-plastic shear failure response of all the beam-to-column connections and allows shear force redistributions among the adjacent connections after any first connection attains its maximum shear resistance. Contribution to the shear resistance from concrete slab is not considered in this study.

![Fig. 4.12 Illustration of shear failure (punching shear) of fire affected floor system](image)

Summarising the above discussion, a multi-failure criterion of joints is proposed in this study, and a joint is deemed to fail when any of the joint failure conditions listed below first occurs.

1) The deformation capacity of any steel joint component is exceeded, based on Table 4.3.

2) An overall deformation limit of 35mm is exceeded for bolt-rows in cleat joints, or a limit of 25mm is exceeded for bolt-rows in end-plate joints.

3) For joints designed in terms of composite behaviour, the rupture deformation of reinforcement bars in accordance with the simplified calculation method proposed by Anderson et al. (2000) is exceeded.

4) The shear force transferred by all joints surrounding a column exceeds the corresponding shear resistance.
4.4 Influence of elevated temperature

Joints in fire exhibit significant reductions in strength and stiffness, and consequently the global stability and robustness of the structure subject to fire can be greatly reduced. Therefore, a method that can predict the actual response of joints subjected to fire has to be carefully implemented. Towards this aim, a ‘reduction factor approach’ is employed to estimate the elevated temperature behaviour of joints, where the reduction factors for strength and stiffness (SRF) are determined. Based on three sets of bare-steel joint tests, Al-Jabri et al. (2004) presented the observed strength and stiffness reduction factors of these joints, as shown in Figure 4.13. It was found that the reduction trends of stiffness and strength obtained in the three tests correlate well with the strength reduction factor recommended by Eurocode 3 (2005b) for carbon steel at strain levels of the proportional limit and 1.0%, respectively. Based on this finding, Ramli-sULONG et al. (2007) proposed new strength and stiffness reduction factors for the material of joints under fire, and employed them for a new joint element developed in ADAPTIC. In this work, the component strength and stiffness reductions factors proposed by Ramli-sULONG et al. (2007) are slight modified, as given in Figure 4.13 and Table 4.4, which demonstrate closer trends compared with the results of Al-Jabri et al. (2004).

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Strength SRF</th>
<th>Stiffness SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td>20°C</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>100°C</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>200°C</td>
<td>0.971</td>
<td>0.807</td>
</tr>
<tr>
<td>300°C</td>
<td>0.941</td>
<td>0.613</td>
</tr>
<tr>
<td>400°C</td>
<td>0.912</td>
<td>0.420</td>
</tr>
<tr>
<td>500°C</td>
<td>0.721</td>
<td>0.280</td>
</tr>
<tr>
<td>600°C</td>
<td>0.360</td>
<td>0.100</td>
</tr>
<tr>
<td>700°C</td>
<td>0.160</td>
<td>0.035</td>
</tr>
<tr>
<td>800°C</td>
<td>0.110</td>
<td>0.020</td>
</tr>
<tr>
<td>900°C</td>
<td>0.060</td>
<td>0.010</td>
</tr>
<tr>
<td>1000°C</td>
<td>0.040</td>
<td>0.005</td>
</tr>
<tr>
<td>1100°C</td>
<td>0.020</td>
<td>0.0025</td>
</tr>
<tr>
<td>1200°C</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>
There is no doubt that the fire degradation properties of an entire joint depend on the elevated temperature behaviour of each individual joint component. Therefore, the reduction factors provided in Table 4.4 can be applied to individual components in the ADAPTIC joint models which are simulated via spring assemblies. To address this, a newly developed spring element, based on piecewise linear degradation approximation of stiffness and strength with temperature, is employed for this study. Under elevated temperature, the spring element can feature a good fit to the reduction factors given in
Figure 4.13. Since the tests conducted by Al-Jabri et al. (2004) were mainly focused on joints under bending action, the degradation properties of joint shear capacity with elevated temperature were not investigated in detail. Therefore, due to the lack of available convincing data, the steel yield strength reduction factors recommended in Eurocode 3 (2005b) are incorporated into the spring representing the joint shear response. It is noted that the current joint model only considers material degradations of joints, whereas the effect of thermal expansion within joints is considered to be negligible.

4.5 Examples of joint modelling

The aim of this section is to present models employing the ADAPTIC component/spring joint elements and to validate these joint models through comparing with selected ambient and fire tests. The previously defined failure criteria are employed to determine the maximum rotation capacity of the joints. The influence of different component post-limit stiffness on overall joint behaviour is also studied. For the joints subject to elevated temperatures, the component degradation characteristics are based on the reduction factors provided in Table 4.4.

4.5.1 Steel flush end-plate joint – Yu et al. (2011)

An experimental program was conducted by Yu et al. (2011) with the aim of investigating the ambient and elevated temperature responses of bare-steel flush end plate joints subject to a combined bending moment and tensile axial force. The combined effect of axial force and bending moment was realised through a specially designed test rig as shown in Figure 4.14. Three load angles (35°, 45°, 55°) and four levels of joint temperature (20°C, 450°C, 550°C, 650°C) were considered. The specimen was heated slowly until the desired temperature, and a gradually increased load was subsequently applied. The test results were given as the resultant force (force in the furnace bar) vs. rotation relationships under the four temperatures. All the tested joints were loaded until failure except the two ambient tests on joints with 35° and 45° load angles due to system malfunction.

The corresponding component/spring model is illustrated in Figure 4.15, where altogether five spring series are considered to represent the three bolt-rows as well as the top and bottom beam flange-to-column contacts. For the three bolt-row spring series,
the axial property in tension is contributed from four components, namely, column web in tension (cwt), column flange in bending (cfb), bolt in tension (bt), and end-plate in bending (epb). The ‘group effect’ is considered for determining the effective widths of the upper two bolt-rows (T-stubs). The compressive characteristic for the five spring series are based on the resistance of column web in compression (cwc), while for the top and bottom outer spring series, the component of beam flange/web in compression (bfwc) is additionally considered. The effect of column web in shear (cws) is ignored in the current model due to relatively strong column and weak end-plate. Four post-limit responses for each ductile component are considered, namely, no strain hardening ($\mu=0$), bilinear response ($\mu=1\%$), bilinear response ($\mu=3\%$), and trilinear response ($\mu_1=3\%$, $\mu_2=1\%$). The material properties for steel beams are $E=176kN/mm^2$, $f_y=356N/mm^2$, and $f_u=502N/mm^2$. For the endplates the material properties are $E=166kN/mm^2$, $f_y=356N/mm^2$, and $f_u=502N/mm^2$. The properties for S355 column steel was not given in Yu et al. (2011), so typical values $E=195kN/mm^2$, $f_y=450N/mm^2$, and $f_u=560N/mm^2$ are employed.
Fig. 4.16 Force-rotation response of end-plate joints under 35° load angle
Fig. 4.17 Force-rotation response of end-plate joints under 45° load angle
The results obtained from the spring models are illustrated in Figures 4.16 to 4.18 for the tests with the load angles of 35°, 45°, and 55°, respectively. Favourable comparisons are found between the spring model predictions and the test results within the elastic range, thus indicating a reliable elastic force-deformation response for each component listed in Table 4.3. However, the post-limit joint behaviour of the spring models can differ greatly with various values of the component post-limit stiffness. It can be seen that under the ambient condition or a relatively low temperature (i.e. 450°C), the spring model with a 1% strain hardening coefficient for its ductile components has the best correlation with the tests, whereas under higher temperatures (i.e. 550°C and 650°C), the tests results are more consistent with the spring models employing components with no strain hardening. In some tests, the resistance even starts to decrease in the plastic range. This implies that low strain hardening coefficients may be assumed for joints
under relatively high temperatures. With respect to the rotation capacity, the test results show that most of the tested connections achieved their maximum resistance at a rotation of around 2°, and they were able to maintain a moderate amount of resistance up to a rotation of 7°. According to the predictions from the component models, the failure rotations for most cases are approximately between 5° to 6°, which offer a reasonable prediction on the conservative side.

4.5.2 Steel flush end-plate joint – Al-Jabri et al. (2005)

A series of elevated temperature tests were conducted by Al-Jabri et al. (2005) to investigate the moment-rotation response of major axis beam-to-column joints, as shown in Figure 4.19. The joint area (i.e. steel connection, column, and a certain length of beams) was exposed to fire with the other parts fully protected. Two groups of flush end-plate steel joint tests are selected to validate the ADAPTIC component joint model, as shown in Figure 4.20. The Group 1 test specimen comprises two 254×102×22UB beams and a 152×152×23UC column connected by 8 mm thick flush end-plates with six M16 Grade 8.8 bolts. The Group 2 test specimen comprises two 356×171×51UB beams connected to a 254×254×89 UC column by 10 mm thick flush end-plates with eight M20 Grade 8.8 bolts. For both of the two groups, loads were applied first, and elevated temperature is applied subsequently. Four connection moments were applied to each specimen, specifically (4kNm, 8kN.m, 13kN.m, 17kN.m) for Group1 and (27kN.m, 56kN.m, 82kN.m, 110kN.m) for Group 2.

Fig. 4.19 Test setup for joints (Al-Jabri et al., 2005)
Based on the elevated temperature joint test results, Al-Jabri et al. (2005) proposed a set of moment-rotation-temperature fitting curves for each test group using a modified Ramberg–Osgood expression (Ramberg and Osgood, 1943), as shown in Figure 4.21. The moment-rotation-temperature curves are employed in this study to compare with the results predicted by the corresponding component models established in ADAPTIC. For the Group 1 joint model, the effective widths of the upper two bolt-rows (T-stubs) are considered as part of a group; for the Group 2 model, the effective widths of the upper three bolt-rows (T-stubs) are considered as part of a group. The material properties for the components are listed in Table 4.5. For each ductile component where a post-limit stiffness is considered, four post-limit stiffness values are compared, namely, no strain hardening ($\mu=0$), bilinear response ($\mu=3\%$), bilinear response ($\mu=5\%$), and trilinear response ($\mu_1=5\%$, $\mu_2=1\%$).
Fig. 4.21 Moment–rotation–temperature curves for end-plate steel joints (Al-Jabri et al., 2005)

Table 4.5 Material properties for specimens (Al-Jabri et al., 2005)

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield stress ((\text{N/mm}^2))</th>
<th>Ultimate stress ((\text{N/mm}^2))</th>
<th>Young’s Modulus ((\text{N/mm}^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1 beam</td>
<td>317</td>
<td>452.5</td>
<td>200000</td>
</tr>
<tr>
<td>Group 1 column</td>
<td>326</td>
<td>455.5</td>
<td>195000</td>
</tr>
<tr>
<td>Group 2 beam</td>
<td>415.5</td>
<td>552</td>
<td>199000</td>
</tr>
<tr>
<td>Group 2 column</td>
<td>426</td>
<td>573</td>
<td>196000</td>
</tr>
<tr>
<td>End plate</td>
<td>322</td>
<td>454</td>
<td>205000</td>
</tr>
<tr>
<td>Bolts</td>
<td>640</td>
<td>800</td>
<td>205000</td>
</tr>
</tbody>
</table>
Fig. 4.22 Moment-rotation relationship for Group 1 tested joints
Fig. 4.23 Moment-rotation relationship for Group 2 tested joints
The moment-rotation relationships of the Group 1 and Group 2 tested specimens predicted by the proposed spring models are shown in Figures 4.22 and 4.23 respectively, where the test data and the fitting curves proposed by Al-Jabri et al. (2005) are also given for comparison. Favourable comparisons are found between the tests and the spring models with the components employing a post-limit stiffness coefficient of approximately between 0.03 and 0.05. The moment resistance obtained from the tests can be underestimated when a strain hardening is ignored. This again indicates that the elastic-perfect plastic component force-deformation response assumed in Eurocode 3 (2005a) may be over-conservative.

4.5.3 Steel double web cleat joint – Yu et al. (2009a, 2009b)

Double web-cleat joint tests are selected here which were also tested by Yu et al. (2009a, 2009b). A combined effect of axial force and bending moment was applied through a specially designed test rig, as illustrated in Figure 4.24, where the geometric configuration of the joint specimen is shown. Three load angles (35°, 45°, 55°) and four levels of joint temperature (20°C, 450°C, 550°C, 650°C) were considered. The specimens were heated slowly until the desired temperatures, and the load was then applied.

The component model for simulating the tests is shown in Figure 4.25, where altogether five spring series are considered to represent the three bolt-rows as well as the top and bottom beam flange-to-column contact areas. For the three bolt-row spring series, the axial property in tension is contributed from six components, namely, column flange in
bending (cfb), bolts in tension (bt), bolts in shear (bs), angle plate in bending (ab), angle plate in bearing (abr), and beam web in bearing (bwbr). For the three inner spring series in compression, bolts in shear (bs), angle plate in bearing (abr), and beam web in bearing (bwbr) are considered. The effective widths of the three bolt-rows (T-stubs) are taken as the smaller value of those considered individually or as part of a group. The compressive characteristic for the two outer spring series are based on the resistance of column web in compression (cwc) and beam web/flange in compression (bwfc). It should be noted that the responses of angle plate in bearing (abr) and beam web in bearing (bwbr) are different in tension and in compression. The ambient material properties for steel beams are \( E=176\,kN/mm^2 \), \( f_y=356\,N/mm^2 \), and \( f_u=502\,N/mm^2 \). For the angle plates the material properties are \( E=134\,kN/mm^2 \), \( f_y=350\,N/mm^2 \), and \( f_u=455\,N/mm^2 \). The properties for S355 column steel were not given in Yu et al. (2009a), so typical values \( E=195\,kN/mm^2 \), \( f_y=450\,N/mm^2 \), and \( f_u=560\,N/mm^2 \) are employed.

Since the bolt-hole diameter is 2mm wider than the bolt diameter (20mm), it is assumed in the model that the bolts are initially located at the centre of the bolt-hole. This is simulated through additional gap-contact elements applied at the three bolt-rows. The friction effect is not considered in this study. Furthermore, two gap-contact elements are applied at the positions of beam top and bottom flanges to consider the 10mm gap between the column flange face and the beam. For the post-limit stiffness, bilinear curves are adopted in this study, and three strain hardening coefficients (0%, 3%, 5%) are employed for the components. The ADAPTIC results are compared with the test results as well as the results from the mechanical model developed by Yu et al. (2009b).
Fig. 4.26 Force-rotation relationship of web cleat joints under 35° load angle
Fig. 4.27 Force-rotation relationship of web cleat joints under 45° load angle
The results obtained from the component models are given in Figures 4.26 to 4.28 for the tests with the load angles of 35°, 45°, and 55°, respectively. It is shown that the ADAPTIC predictions compare well with the test results, although for some cases the rotation capacity the joint is underestimated, particularly at ambient conditions. This is due to the limit of 35mm maximum bolt-row tensile deformation capacity which can be conservative. The discrepancies of the results may also be attributed to the neglect of friction in the component model. In addition, it is assumed in the model that the load angle remains unchanged during the loading procedure, while during the test it was found that the load angle can be changed by ±3°.
4.5.4 Composite end-plate joint – ROBUSTFIRE project (2011)

Under a typical localised fire scenario where fire occurs near a column, the joint directly exposed to the fire can be subjected a significant sagging bending moment subsequent to column buckling. In order to evaluate the ductility supply of the fire affected joint after the loss of the column, a test programme, with the setup illustrated in Figure 4.29, was proposed as a part of the European RFCS ROBUSTFIRE project, where emphasis is given to the robustness of car parks against localised fire. The test specimens were designed according to a standard open car park structure specially designed for the ROBUSTFIRE project (Gens, 2010), and this building was deemed to be a typical car park type in Europe. The tested frame was comprised of two unprotected 3m length composite beams with IPE550 steel cross-sections, Grade S355, and one unprotected HEB300 cross-section steel column, Grade S460. The steel beams are fully connected to a 130 mm thickness composite slab through fully composite shear studs. Flush end-plates are employed with eight M30 Grade 10.9 steel bolts. The geometric properties of the tested frame are given in Figure 4.30 (Haremza et al., 2011).
Two tests (Test 2 and Test 3) are selected to compare with the corresponding component joint models established in ADAPTIC, as shown in Figure 4.31. For both of the tests, the joint was first heated to a stabilised peak temperature, then the column base was gradually relaxed and subsequently an increasing downward vertical point load was applied at the top of the HEB 300 column. No axial restraint was applied at the beam ends throughout the entire test. The maximum temperatures in the bottom flange of the steel beam were 500°C and 700°C for Tests 2 and 3 respectively, and the temperature was kept unchanged during the loading procedure. However, the temperature distribution was not uniform in the entire joint area, and different maximum temperatures were found at different parts of the joint, as given in Table 4.6. Accordingly, for the component joint model established in ADAPTIC, various temperatures are applied onto different joint components during the loading procedure.
Table 4.6 Peak temperatures of joint components

<table>
<thead>
<tr>
<th>Location</th>
<th>Test 2</th>
<th>Test 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column flange</td>
<td>400°C</td>
<td>483°C</td>
</tr>
<tr>
<td>Column web</td>
<td>470°C</td>
<td>565°C</td>
</tr>
<tr>
<td>End-plate</td>
<td>430°C</td>
<td>529°C</td>
</tr>
<tr>
<td>Beam web</td>
<td>470°C</td>
<td>620°C</td>
</tr>
<tr>
<td>Beam flange</td>
<td>500°C</td>
<td>700°C</td>
</tr>
<tr>
<td>Bolt</td>
<td>390°C</td>
<td>505°C</td>
</tr>
<tr>
<td>Concrete</td>
<td>180°C</td>
<td>216°C</td>
</tr>
</tbody>
</table>

For the four inner bolt-row spring series of the component model, the axial property in tension is contributed from four components: column web in tension (cwt), column flange in bending (cfb), bolt in tension (bt), and end-plate in bending (epb). The effective width of the T-stub is selected with the bolt-rows considered as part of a group. The compressive characteristic for all five spring series are based on the resistance of the column web in compression (cwc). For the top and bottom outer spring series
representing the contact locations between the beam flanges and the column flange, the resistance of beam flange/web in compression (bfwc) is additionally considered. The effect of column web in shear (cws) is ignored due to the symmetry of the tested frame.

Three types of post-limit responses for each ductile component in the steel connection are considered, namely, no strain hardening (μ=0), bilinear response (μ=3%), and bilinear response (μ=5%). Since the concrete is mainly under compression, the concrete slab is simulated via beam-column elements neglecting the ribs and the steel deck. Rigid links are employed to connect the steel beam and the concrete to consider a full shear interaction. The material properties for the flange and the web of steel beams are $E = 205 \text{kN/mm}^2$, $f_y = 395.7 \text{N/mm}^2$, $f_u = 516.7 \text{N/mm}^2$, and $E = 205 \text{kN/mm}^2$, $f_y = 432.3 \text{N/mm}^2$, $f_u = 538.7 \text{N/mm}^2$, respectively. For the endplates the material properties are $E = 205 \text{kN/mm}^2$, $f_y = 395.7 \text{N/mm}^2$, and $f_u = 516.7 \text{N/mm}^2$. The properties for the S460 column flange and web are $E = 205 \text{kN/mm}^2$, $f_y = 515.7 \text{N/mm}^2$, $f_u = 599 \text{N/mm}^2$, and $E = 205 \text{kN/mm}^2$, $f_y = 503.7 \text{N/mm}^2$, $f_u = 571.3 \text{N/mm}^2$, respectively. The compressive strength for concrete is $f_c = 33 \text{N/mm}^2$.

The bending moment vs. rotation relationships predicted from the component models are illustrated in Figures 4.32 and 4.33 for Tests 2 and 3, respectively. Favourable comparison is observed for Test 2 regarding initial stiffness, and the moment capacity is predicted well when a 5% strain hardening is employed for the component post-limit stiffness. The sudden reduction of moment resistance at a rotation of around 30 mrad is due to compressive crushing of concrete. For Test 3, the moment capacity is well predicted, but the initial stiffness is overestimated by the component model, although the predicted initial stiffness is close to the observed joint stiffness during the reloading process of the test. The moment capacity observed in the test is predicted well by the component model when a strain hardening of 3% is employed for the post-limit stiffness.

Furthermore, it can be seen that the ductility supply / maximum rotation of the joints in both tests are underestimated by the component model, particularly for Test 3. This is due to the fact that the failure modes of the component models for both of the tests are governed by tensile rupture of the lowest bolt-row (furthest from the centre of compression), where the allowed maximum bolt-row elongation of 25mm is exceeded. This again indicates that the joint failure criteria defined in Section 4.3 can be quite
conservative, though these will still be adopted for the rest of this thesis in the absence of further supporting experimental results.

Fig. 4.32 Moment-rotation relationship of Test 2 joint

Fig. 4.33 Moment-rotation relationship of Test 3 joint

### 4.6 Summary and conclusions

This chapter has discussed the main component joint modelling techniques which can be employed in overall structural modelling frameworks. Emphasis is given to the current codified design procedure and its extension to different joint types (e.g. web cleat joint) and elevated temperature conditions. Importantly, failure criteria of joints
are proposed, which are essential to the investigation of the overall structural ductility supply which determines resistance to progressive collapse. Based on the predefined failure criteria and the elastic characteristics (i.e. initial stiffness and yield resistance) of the components provided in Eurocode 3 (2005a) and by other researchers, component joint models which are based on ADAPTIC spring and rigid link elements are developed. Finally, available ambient and fire joint tests are selected to validate the component models, and conclusions are drawn as follows:

- Favourable comparisons are found between the results obtained from the tests and the corresponding component models, particularly in terms of initial joint rotation stiffness under bending or combined bending-axial actions. This is essential for reliable predictions of joint performance under extreme loading, where significant axial forces may develop during the compressive arching or catenary stages. Furthermore, elevated temperature effects are considered via degradation component properties, a strategy shown to offer good accuracy.
- The post-limit stiffness of ductile components has a significant influence on the elasto-plastic response of the considered joints. In general, assuming an elastic-perfectly plastic component force-deformation response (i.e. no strain hardening) can lead to an underestimation of the moment capacity. According to the findings of the joint tests considered for this study, a strain hardening coefficient of between 1% and 3% for ductile components typically yields favourable predictions for most cases.

With the ductility supply of joints considered in terms of rotation capacity, the joint multi-failure criteria defined in this study are found to be conservative in most cases, especially for joints under elevated temperature conditions where steel may be more ductile than that at ambient temperature. Towards a more clear understanding of the rotation capacities of various types of joint, further experimental investigations are needed.
CHAPTER 5

Robustness of Structures under Localised Fire

5.1 Introduction

Numerical analysis can give a clear and relatively accurate insight into the behaviour of structures subject to fire. With sufficient confidence of modelling the individual structural components, e.g. beams, columns, floor systems, and joints, as discussed in the previous chapters, this chapter performs a detailed numerical study on a reference structure subject to a proposed localised fire scenario, where two main aims are sought. The first aim is to capture the response of the reference structure under localised fire as accurately as possible with a comprehensive consideration of structural ductility supply for robustness assessment. Detailed structural analysis is performed through employing a realistic temperature distribution obtained from detailed heat transfer analysis. The second aim is to employ the structural behaviour obtained from detailed thermal and structural analysis as a benchmark for the verification of the simplified Temperature-Dependent Approach (TDA) and the Temperature-Independent Approach (TIA) which are presented in the next chapters.

In the study undertaken, the detailed structural configuration of the reference building is introduced first in Section 5.2. This is followed by presenting the selected fire scenario as well as the corresponding temperature distributions within the structural members obtained from thermal analysis, which are introduced into the ADAPTIC structural model for further structural analysis. In Section 5.4, the structural modelling techniques of the reference building are elaborated, where a multi-level modelling approach with
various levels of simplifications is proposed. Dynamic analysis is performed to evaluate potential dynamic effects, particularly during column buckling. Subsequently in Section 5.5, failure criteria of the overall structural system are defined, and a ‘Robustness Limit State’ as well as a robustness assessment procedure is proposed accordingly. Finally, the robustness of the reference building as obtained from the structural analysis is discussed, and possible joint failure modes that could potentially trigger progressive collapse are outlined.

### 5.2 Configuration of reference structure

A typical eight-storey car park designed for the ROBUSTFIRE project (Gens, 2010) is considered here as the reference building, as illustrated in Figure 5.1(a). Conventional composite steel-concrete construction is selected. The roof is designed identically to the other floors because it is also used for parking. IPE 550 beams, 10m in length, and IPE450 beams, 16m in length, are used for the primary beams and secondary beams respectively for all floors. The height of each floor is 3 m, which makes a total height of the building equal to 24m. The size of the columns varies at every two floors from the HEB220 size for the top two floors to the HEB550 size for the bottom two floors, as shown in Figure 5.1(b). The column bases are assumed to be fixed, and lateral resistance is provided by wind-bracings and the concrete ramp on each side of the building, which makes a non-sway structural type.

![Structural layout of reference car park: (a) 3D frame skeleton, (b) geometric dimensions](image)

*Fig. 5.1* Structural layout of reference car park: (a) 3D frame skeleton, (b) geometric dimensions (cont’d…)*
Type ‘COFRAPLUS 60’ ribbed slab (Gens, 2010) with steel deck is selected for the composite floor system with a 3.333m span, as illustrated in Figure 5.2. The floor slab is 120mm deep, and consists of a ribbed metal sheet of 1 mm thickness. The steel deck not only acts to resist gravity loading in the longitudinal direction of the ribs, but also performs as a formwork during the casting of the concrete. A basic mesh of \( \Phi 8 \)mm spaced at 200mm (\( A_s = 250 \text{mm}^2/\text{m} \)) is used in order to limit concrete cracking. To enhance the composite action for the major axis beam-to-column joints, additional rebars (10×\( \Phi 12 \)) are added in order to increase the bending resistance of these joints. Full shear interaction between concrete and steel are assumed for the floor system. All the floors including the roof are designed to carry the same gravity load. The values of the unfactored composite slab self-weight and vehicle loads on each floor are 2.145kN/m² and 2.5kN/m², respectively. Accordingly, a basic load of 5kN/m² is considered for this car park.
Semi-rigid joints are employed for the reference car park. Figure 5.3 illustrates the geometric configuration of the joints according to the design undertaken by Gens (2010). Flush end-plate connections are employed for the major axis beam-to-column joints, as shown in Figure 5.3(a). The S355 15mm-thick end-plate with four bolt-rows is welded to the end of the supported beam. Eight M30 10.9 bolts in 32mm-diameter holes are employed to connect the plate to the column flange. A stiffener plate is added in the steel column to avoid premature yielding of the column web under compression. The considered joints are assumed to be double-sided and have a shear capacity of 1302kN governed by the shear failure of beam web. Double web-cleat (double angle) connections using four rows of M24 10.9 bolts are adopted for the minor axis beam-to-column joints as well as the beam-to-beam joints, as shown in Figure 5.3(b) and 5.3(c). A 10mm gap exists between the beam and the connected column or beam. This joint has a shear capacity of 617kN governed by the bearing failure of the beam web. All The joints are designed in accordance with the current Eurocode guidelines (Eurocode 3, 2005a). In addition, as far as structural integrity is concerned, the tying force requirements specified in British Standard (BSI, 2001) are satisfied.
Fig. 5.3 Geometric configuration of joints: (a) major axis beam-to-column joint, (b) minor axis beam-to-column joint, (c) beam-to-beam joint
5.3 Selected fire scenario and temperature distribution

It is assumed that four ‘V3 class’ cars are parked around an internal column, and the fires are triggered in the sequence as shown in Figure 5.4. The interval of the fire spreading from one car to another/other car/s is 12 minutes, and the history of the heat flux on the surfaces of the structural members (e.g. steel beams and composite slab) are captured at 3 minute intervals using the method of Hasemi et al. (1995). The rate of heat release of each burning car is given in Figure 5.5, where the maximum heat release rate is 8.3MW. The vertical distance between the fire origin and the ceiling is 2.4m.

Based on the proposed fire scenario, thermal analysis was conducted by Obiala et al. (2010), where the finite element programme SAFIR (Franssen, 2005) was employed, as illustrated in Figure 5.6. The thermal output data are then extracted and input into the
structural model established in ADAPTIC (Izzuddin, 1991) for structural analysis. Here, several temperature distribution results in important structural members, as highlighted in Figure 5.4, are briefly presented and discussed.

![Illustration of thermal analysis results](image1)

**Fig. 5.6** Illustration of thermal analysis results

The temperature-time curves at the position of the beam-to-column joint and the two adjacent beam-to-beam joints (‘beam-beam joint 1’ and ‘beam-beam joint 2’ as marked in Figure 5.4) are shown in Figure 5.7. An evenly distributed temperature field is assumed within each joint. It is observed that the peak temperature within the beam-to-column joint is around 750°C at the time of 30 minutes. This high temperature can cause a considerable reduction to the resistance of the fire affected joints and slab system. The temperature distribution within the fire affected column is considered to be uniform and is identical to the one within the beam-to-column joint. This assumption can be conservative since in reality lower temperatures can be found at the column base. However, this is acceptable since emphasis of this study is mainly given to structural robustness of the double span floor system after column loss, so the exact temperature distribution in the column may not be necessary.

![Temperature-time curves for joints](image2)

**Fig. 5.7** Temperature-time curves for joints
Figure 5.8 shows the temperature distributions along the primary beam (marked in Figure 5.4) obtained by SAFIR, where three points across the cross-section (i.e. top flange, web, and bottom flange) are considered along the 20m double-span. Four time clips are selected and shown, where the first three (27mins, 33mins, 39mins) are the instants close to the peak temperature and the fourth (60mins) reflects a cooling phase. Based on the results, the peak temperature of approximately 750°C is found in the beam web, and a distinct thermal gradient over the depth of the beam cross-section is observed. In addition, due to a relatively thin beam web employed, the temperature in the mid-depth beam web is slightly higher than that in the bottom beam flange, and this is in line with the test results discussed in Chapter 3 (Wainman and Kirby, 1988). At the peak temperature, the temperatures at the bottom flange and web are considerably higher than that at the top flange, whereas this difference decreases during the cooling phase, where the temperature distribution tends to be even over the cross-section. Similar trends are observed from the results of the temperature distribution within the selected secondary steel beam, as shown in Figure 5.9, where temperatures at three
points across the cross-section are captured along the 32m double-span. The peak temperature found within the secondary beam is around 920°C in the web. For both primary and secondary steel beams, high temperatures are only observed within parts of the beam length in the vicinity of the fire origins, whereas for adjacent beam parts which are not immediately above the fire origin, the temperature decreases rapidly to room temperature. This indicates that the fire affected area is rather localised, and the surrounding structural members may be assumed to be at much lower temperatures.

Figure 5.10 provides the temperature distribution within the concrete slab above the primary beam (as marked in Figure 5.4), where temperatures at five points over the cross-section of the ribbed slab (i.e. steel deck 1, steel deck 2, concrete top, concrete middle, and concrete bottom) are captured along the 20m double-span. It is observed that during the heating phase, the temperature in the steel deck is much higher than that in the concrete, even for the concrete immediately above the steel deck. The temperature decreases further towards the top of the slab which is almost under room temperature. During the cooling phase, a faster temperature drop is found in the steel
deck than in the concrete, thus reducing the thermal gradient over the cross-section. Also, high temperatures are only observed within the slab near the fire origin, while the surrounding parts of the slab remain under a much lower temperature.

![Fig. 5.10 Temperature distribution within concrete slab above primary beam](image)

5.4 Structural modelling

5.4.1 Multi-level substructure modelling approach

A multi-level modelling approach has been developed for assessing the robustness of multi-storey buildings subject to localised fire, which draws on and elaborates the approach proposed by Izzuddin et al. (2008) for sudden column loss. Three modelling levels (A, B and C) can be generally employed, as shown in Figure 5.11. At level A, consideration is given to a whole system of the influenced sub-structure with appropriate boundary conditions representing the surrounding cool structures. Provided that the upper ambient floor systems have similar structure type and applied loading, the assessment model can be simplified to level B, where a reduced model consisting of the
fire affected floor-column system and the upper ambient floor system are considered. At this level, the two systems (i.e. fire and ambient) are investigated separately. The derived characteristics of the ambient floors can be incorporated into a nonlinear ‘spring’ applied at the top of the fire-affected floor system. Then emphasis is given to the behaviour of the fire affected floor system with the added spring. Finally, at level C, planar effects within the floor slab are ignored, and grillage models with composite beams are considered instead. The effective width of the slab flange can be obtained through Eurocode 4 (2004) considering shear lag, and the ribs can be ignored in the slab flange along the perpendicular direction.

To obtain the response of the spring representing the upper ambient floors, a 5kN/m² UDL and an upward point load (exerted at the position of internal column) are initially applied to the ambient floor model. The upward point load is used to represent the supporting internal column, such that the initial floor deflection at the column location is zero. Subsequently, an increasing downward point load is exerted at the same position until one of the joints exceeds its ductility supply. Considering that a fire affected column will be subjected to thermal expansion, the evaluation of the upward stiffness of the ambient floor is also necessary. The rotational stiffness of the spring can be calculated as $4E_c I_c / L_c$, where $E_c$, $I_c$, and $L_c$ are respectively the young’s modulus, the
second moment of area, and the length of the column immediately above. For both model levels B and C, the initial load from upper ambient floors is represented by a point load applied on top of the fire affected floor with value taken as $0.25P_{gravity}$ (Vlassis et al., 2008; Izzuddin, et al., 2008), where $P_{gravity}$ is the overall gravity loading applied on the upper ambient double-span floors.

In order to verify the accuracy of the proposed model reduction procedure, a four-storey steel-framed composite structure is established in ADAPTIC, as shown in Figure 5.12(a). Joint details are ignored in the model, and beams and columns are assumed to be rigidly connected. All the boundary springs are considered as fully restrained. It is assumed that the internal column at the ground floor experiences a uniform increase in temperature, and this temperature reduces linearly from the column top to room temperature 3m away from the column within the connected steel beams. The slab is considered as fully protected and remains under ambient conditions throughout the heating procedure. This model is then reduced to a level C model comprising the fire floor system and an additional spring, as shown in Figure 5.12(b). The downward and upward stiffness of one ambient floor is given in Figure 5.13. In ADAPTIC, the spring is modelled using an element that is capable of simulating the force-displacement relationships with multi-linear approximations. The loads exerted onto the reduced model include a $5\text{kN/m}^2$ UDL, a point load from the upper three ambient floors and the subsequent thermal load.

![Fig. 5.12 Deflection shape of ADAPTIC model with internal column at 1000°C: (a) 4-storey model, (b) reduced system model](image_url)
CHAPTER 5 Robustness of Structures under Localised Fire

Fig. 5.13 Response of one ambient floor

Fig. 5.14 Variation of vertical displacement at column top

Fig. 5.15 Variation of column axial force
Figure 5.14 presents the vertical displacement at the column top for the 4-storey model as well as the reduced system model, while Figure 5.15 provides the column axial force ratio $P_t/P_0$ from the two models, where $P_t$ is the axial force of the heated column, and $P_0$ is the initial column axial force under the ambient condition. The peak column axial force ratio $P_t/P_0$, the maximum floor deflection, and the critical temperature of the two levels of the models are listed and compared in Table 5.1, where the critical temperature is defined at the instant when the column internal axial force returns back to its initial value under ambient temperature. It is found that the responses of the two models are very close, which verifies that a reduced system model is capable of capturing the behaviour of the structures subject to localised fire with sufficient accuracy compared with more complex multi-storey models, provided that the response of upper ambient floors is accurately modelled. Therefore, in the following study, reduced substructure models (Levels B and C) are employed for simulating the reference building, as shown in Figure 5.16.

**Table 5.1** Comparison of data between two models with internal column under fire

<table>
<thead>
<tr>
<th>Structural response</th>
<th>Full 4-storey model</th>
<th>Reduced system model</th>
<th>Discrepancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum column axial force ratio $P_t/P_0$</td>
<td>1.195</td>
<td>1.172</td>
<td>1.92%</td>
</tr>
<tr>
<td>Vertical deflection at 1000°C(mm)</td>
<td>143.5</td>
<td>148.9</td>
<td>3.76%</td>
</tr>
<tr>
<td>Critical temperature (°C)</td>
<td>601.8</td>
<td>602.5</td>
<td>0.12%</td>
</tr>
</tbody>
</table>

**Fig. 5.16** ADAPTIC substructure models: (a) Level B full slab model, (b) Level C grillage model
5.4.2 Material modelling

The material modelling technique for the reference car park is similar to that considered in Chapter 3 for the individual structural member models (e.g. steel/composite beams, steel columns, and slabs). For steel, different steel grades are employed for beams, steel decks, columns, and reinforcement bars. The ambient steel properties for the structural members are given in Table 5.2. In ADAPTIC, a linear-elliptic material model is used for steel under ambient and elevated temperature conditions. The material degradations (i.e. yield strength $f_y$, proportional limit $f_p$, and elastic modulus $E$) and the thermal strain of steel induced by elevated temperature are based on the relative recommendations specified in Eurocode 3 (2005b).

Table 5.2 Ambient properties for steel

<table>
<thead>
<tr>
<th>Component</th>
<th>Yield strength $f_y$ (N/mm$^2$)</th>
<th>Elastic modulus $E$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>355</td>
<td>210000</td>
</tr>
<tr>
<td>Reinforcement bar</td>
<td>500</td>
<td>200000</td>
</tr>
<tr>
<td>Column</td>
<td>460</td>
<td>210000</td>
</tr>
<tr>
<td>Steel deck</td>
<td>350</td>
<td>210000</td>
</tr>
</tbody>
</table>

For concrete, an advanced nonlinear concrete material model, which accounts for the combined effects of compressive nonlinearity, tensile crack opening and closure, and temperature, is employed for the 2D slab employing shell elements (Level B model). Details of this concrete material model have been introduced in Chapter 3 for 2D slab modelling, and are elaborated in more detail in Izzuddin et al. (2004). The basic ambient properties of concrete are listed in Table 5.3. The material degradations and the thermal strain of concrete induced by temperature elevation are in accordance with the relative recommendation in Eurocode 4 (2005). It is noted that the tension resistance of concrete is also considered, allowing for linear softening after cracking. On the other hand, the stress-strain relationship of the ‘concrete flange’ for the Level C model with grillage approximations is slightly different, where a linear descending (softening) branch in compression is considered instead of a nonlinear one used in the 2D slab concrete material model. Details of the concrete material model for 1D beam-column elements have also been presented in Chapter 3 for the 1D composite beam modelling and can be found in more detail in the ADAPTIC user manual (Izzuddin, 2010).
illustrations of the stress-strain relationships for steel and concrete (Level B model) are provided in Figure 5.17.

<table>
<thead>
<tr>
<th>Table 5.3 Ambient properties for concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Young’s modulus E (N/mm²)</strong></td>
</tr>
<tr>
<td><strong>Poisson’s ratio</strong></td>
</tr>
<tr>
<td><strong>Tensile strength f_t (N/mm²)</strong></td>
</tr>
<tr>
<td><strong>Compressive strength f_c (N/mm²)</strong></td>
</tr>
<tr>
<td><strong>Compressive strain ε_c1</strong></td>
</tr>
<tr>
<td><strong>Tensile softening slope E_cr (N/mm²)</strong></td>
</tr>
<tr>
<td><strong>Normalised initial compressive strength s_c</strong></td>
</tr>
<tr>
<td><strong>Normalised residual compressive strength r_c</strong></td>
</tr>
</tbody>
</table>

**Fig. 5.17** Illustrations of steel and concrete stress-strain relationships

5.4.3 Beam/column modelling

Cubic elasto-plastic beam-column elements are employed to model the steel beams and columns (Izzuddin and Elnashai, 1993). Full shear interaction between the steel beams and concrete slabs is considered and realised by interconnecting the steel beams with the slab using rigid links. A linear elastic boundary spring is applied at the end of each primary beam to represent the restraint from adjacent members. The stiffness of the boundary spring allows for different tensile and compressive responses. The stiffness of the boundary spring in tension is contributed by the axial stiffness of the adjacent beam $E_sA_s/L_b$ and the axial stiffness of the steel connection on the other side of the supported column, where $E_s$ is the steel Young’s modulus, $A_s$ is the area of the steel beam cross...
section, and $L_b$ is the length of the adjacent beam. Due to cracking, the tensile stiffness contributed by the adjacent concrete is ignored. The compressive stiffness of the primary beam boundary spring is calculated from the axial stiffness of the composite cross-section $E_c A_c/L_b + E_c A_c/L_b$ assuming an effective width of the adjacent concrete flange, where $E_c$ is the concrete Young’s modulus, and $A_c$ is the effective cross-section of concrete flange ignoring the ribbed part. The rotational flexibility of the boundary spring is ignored, thus full rotational restraint is assumed. In the considered model, the stiffness of the primary beam boundary springs in tension and compression are 229.7kN/mm and 734.2kN/mm, respectively. The stiffness of the boundary springs for the secondary beams is idealised as zero due to the lack of axial restraint in the transverse direction of the car park as well as the limited horizontal stiffness provided by the façade column in bending.

### 5.4.4 Slab modelling

Regarding the selected model levels B and C, two slab modelling approaches are considered, which are named full 2D slab model and grillage slab model respectively.

For the level B full slab model, shell elements are employed (Izzuddin et al., 2004). Additional rib freedoms in conjunction with the conventional freedoms typically associated with shell elements are incorporated in the shell element which was shown to provide sufficient accuracy and numerical robustness (Elghazouli and Izzuddin, 2004a). The slabs are assumed to be vertically supported and free to rotate along supporting edges. Full horizontal (planar) restraints are applied along the edges of the slab perpendicular to the primary beam directions, while no horizontal restraint is applied along the two other edges. Additional reinforcement bars are positioned in the slab to mobilise the composite response. This is realised by employing the elements locally with larger reinforcement ratios in the required direction in the vicinity of joint areas where additional rebars are applied.

With respect to the level C slab model, a composite slab grillage is established. The slab is represented by elasto-plastic beam-column elements, of which the calculation of the effective width for the concrete slab is provided by the Eurocode 4 (2004) to consider the shear lag effects. Assuming a distance between the centres of the two outstand shear connectors of 150mm, and considering a double-span action of the composite beams,
the effective concrete flange width for the secondary beams at both mid-span and support areas is 3.333m, while for the primary beams at mid-span and the two end-supporting areas this becomes 3.65m and 2.025m, respectively. Instead of adopting a linearly increasing width within the end-supporting area, a constant average value of effective width (i.e. the mean value of 3.65m and 2.025m) is employed. The beam-column elements with a T-section are employed for simulating the primary beam concrete sections parallel to the ribs. The ratio of upper width over lower width of the T-section is determined by the ratio of total slab width over the width of ribs. In addition to the anti-crack reinforcement mesh (250mm²/m), further equivalent reinforcement is added at the bottom of the elements to simulate the steel deck (1000mm²/m). For the secondary concrete sections perpendicular to the direction of ribs, only the concrete and reinforcement above the top of rib profile is considered, and the contribution of the steel deck is ignored. The primary and secondary composite sections are illustrated in Figure 5.18. For both full slab model and grillage model, equivalent lumped masses are applied at the nodes of the elements modelling the slab to account for inertia effects in dynamic analysis.

![Fig. 5.18 Illustrations of beam cross-sections in grillage model](image)

At elevated temperature, the temperature distribution over the cross-section is modelled using a multi-linear approximation through several monitoring points for the Level B full slab model, as shown in Figure 5.19. The temperature at the middle position of the ‘concrete cover’, the temperature difference between the cover bottom and slab top, and the temperature within the steel deck of the cover part of slab are considered. For the ribbed part, the temperature of the concrete rib and that within the steel deck at the rib bottom are additionally considered.
For the grillage model (Level C), the elevated temperature modelling of the slab flange is similar to that used for the steel beams. For the primary beam slab flange, three temperatures (slab top, slab middle, steel deck) at each node are considered. Parabolic/quadratic temperature curves are employed over the depth of the cross-section according to the three points. For the secondary beam slab flange, with the steel deck and ribs ignored, the remaining parts are generally under room temperature, so no elevated temperatures are considered.

5.4.5 Joint modelling

The component method as well as the failure criteria discussed in Chapter 4 are employed here for joint modelling. As introduced previously, the underlying principle of the component method is to separate a joint into a tension zone (T-stubs), a compression zone, and a shear zone, and each zone is further divided into several basic components of known characteristics. In ADAPTIC, the bolt-rows are represented by discrete spring elements, and these are connected by rigid links. The axial springs are defined in terms of stiffness and resistance which are obtained from the characteristics of the corresponding components. In the current model, bi-linear curves are employed to model the force-deformation response of each component.

Figure 5.20 presents the component model developed for the major axis flush end-plate joints. For the four internal bolt-row spring series, the axial property in tension is contributed from four components, namely, column web in tension (cwt), column flange in bending (cfb), bolt in tension (bt), and end-plate in bending (epb). The compressive characteristic for the inner springs is determined by the resistance of the column web in compression (cwc). The two outer spring series are free to be pulled in tension, but in
compression the resistance and stiffness are contributed from the beam web/flange in compression (bwfc). Where column web stiffeners are added, the stiffness of the column web in compression (cwc) can be considered as infinite. Moreover, for single-sided joints, the column web in shear (cws) is represented by applying an additional spring at the bottom flange of the beam.

![Component model for major axis beam-to-column joint](image)

**Fig. 5.20** Component model for major axis beam-to-column joint

Minor axis beam-to-column and beam-to-beam joints are mainly designed for resisting shear force, where composite action is not considered in the design. Figure 5.21 illustrates the mechanical model of the minor axis beam-to-column and beam-to-beam joints. The axial property in the tension zone is contributed from five components, namely, bolts in tension (bt), bolts in shear (bs), angle plate in bending (ab), angle plate in bearing (abr), and beam web in bearing (bwbr). The compressive characteristics for the four inner spring series depend on the properties of bolts in shear (bs), angle plate in bearing (abr), and beam web in bearing (bwbr). The two outer springs are free to be pulled in tension, but in compression the resistance and stiffness are contributed from the beam web/flange in compression (bwfc) and column web in bending (cwb). Moreover, two additional contacts element are adopted to model the 10mm gap between the beam flanges and the column/beam web. The gap between bolts and bolt-holes and the slip friction between bolts and plates are not considered in this component model.
5.5 Robustness assessment of reference structure

5.5.1 Definition of structural failure and robustness limit state

A structural robustness assessment procedure is applied here to the considered car park under localised fire, with the previously discussed temperature distributions and structural idealisations, utilising the joint failure criteria defined in Chapter 4. Recent research (Vlassis et al., 2009) indicated that the collapse of even one floor can cause severe damage on the floor below for typical existing steel-framed composite constructions, thus triggering progressive collapse. Therefore, the definition of safe structure in this study is based on the avoidance of collapse in any of the affected floors. In other words, structural failure/progressive collapse occurs when the deformation of either the fire affected floor or the upper ambient floors exceeds their respective ductility capacity. In this respect, the failure of any floor system is attributed to the ductility failure of any surrounding ambient joint on that floor, thus failure criteria are defined in terms of whether the ductility limits of the joints are exceeded. If the surrounding ambient joints have sufficient resistance, but the joints directly exposed to fire fail first, the structure is still deemed safe. The proposed failure assessment procedure is given in Figure 5.22. It should be noted that it is possible for a structure to survive after first failure of the surrounding ambient joints, provided that sufficient resisting mechanism (e.g. membrane action) is maintained by the slab. However, residual load resistance beyond the failure of the surrounding ambient joints has not been fully studied and thus needs further investigation that is beyond the scope of the present work. Accordingly, the defined failure criteria in this study should be on the
conservative side. Based on the definition of structural failure, the load-deflection response of the spring representing one typical ambient floor with the 2D full slab and the grillage slab is given in Figure 5.23, where the failure mode of the ambient floor systems is found to be governed by the rupture of ambient joint rebars under hogging moment.

Perform thermal analysis and apply fire loading

Run structural analysis

No joint failure observed at the end of the fire

Structure safe

First joint failure occurs

First failure occurs at surrounding ambient joints (either fire or ambient floor)

Progressive collapse triggered

First failure occurs at fire affected joints

Lead to successive failure of surrounding ambient joint?

Yes

Progressive collapse triggered

No

Structure safe

Fig. 5.22 Robustness assessment procedure of car parks subject to vehicle fire

Fig. 5.23 Ductility supply of spring representing one ambient floors
5.5.2 Structural robustness – fire at floor level 1

The deflection of the fire affected floor and the column axial forces for the grillage model and the full slab model during the considered fire scenario at the ground floor are shown in Figures 5.24 and 5.25 respectively. The variation of the column axial force is represented by a $P_t/P_0$ ratio, where $P_t$ is the column axial force at elevated temperature during the fire, and $P_0$ represents the initial compressive force carried by the column. The structural behaviour predicted by the 2D full slab model shows that buckling of the fire affected column occurs at around 25 minutes (column temperature of around 600°C), but this does not directly lead to overall failure of the system, which is largely attributed to the additional resistance provided by the seven ambient floors above the fire affected floor; therefore, the vertical floor deflection is arrested and then stabilized at approximately 300mm without exceeding the ductility supply offered by joints. The corresponding stabilised deflected shape after column buckling of floor level 1 is illustrated in Figure 5.26.

After column buckling, shear failure (punching shear) of the fire affected beam-to-column joint is observed at a time of 30 minutes. The corresponding failure temperature of the joint and column is 741°C, under which condition the overall shear force resisted by the four fire affected beam-to-column steel connections exceeds their overall elevated temperature shear capacity, as explained in Figure 5.27. In the current mode, punching shear is simulated artificially by abruptly removing the resistance of fire affected connections completely. Figure 5.27 shows that before the buckling of the column, the fire affected beam-to-column connections are subjected to significant shear forces, but no shear failure is found due to the limited reduction of their shear capacity. When the column buckles, the upper ambient floors have sufficient vertical load resistance to ‘pull up’ the fire affected floor with a stabilized deflection of 300mm, such that in effect the buckled column can be seen to continue to provide the vertical resistance. In this case the shear force transferred to the upper ambient structure through the steel beam-to-column connections stabilizes at around 600kN (but less than the initial ambient value of 800kN). As the temperature keeps increasing, the shear capacity of the fire affected beam-to-column joint is further reduced, and when it drops below the transferred shear force, first joint failure of the system is triggered by means of punching shear. Initiated by the joint shear failure, the fire affected floor is detached from the upper ambient floors. The final deflection of the individual fire affected floor
is then arrested at 740mm but still with sufficient ductility supply provided by the surrounding ambient joints. Since no successive joint failure is observed after the first joint failure (punching shear), progressive collapse of the structure is prevented.

![Deflection of fire affected floor for fire at floor level 1](Fig. 5.24)

![Axial force of fire affected column for fire at floor level 1](Fig. 5.25)

![Deflected shape of floor level 1 at time = 25m00s](Fig. 5.26)
Fig. 5.27 Shear response of fire affected joint for fire at floor level 1 (full slab model)

On the other hand, the grillage model predicts a less desirable robustness response. Figure 5.28 illustrates the first failure and the successive collapse mode of the grillage model subject to vehicle fire affecting floor level 1. First joint failure is observed at a time of 25 minutes and 27 seconds in the fire affected major axis beam-to-column joint under sagging moment along with the buckling of the fire affected column, where the elongation of the lowest bolt-row (furthest from the centre of compression) exceeds the maximum limit of 25mm. Soon afterwards, successive joint failure is directly triggered in the surrounding ambient major axis beam-to-column joints due to the ruptures of reinforcement rebar. This progressive collapse mode is a typical ‘double-span’ failure mechanism which is largely due to the buckling of the fire affected column and insufficient upper ambient floor resistance.

Fig. 5.28 First and successive joint failures of grillage model for fire at floor level 1
5.5.3 Structural robustness – fire at floor level 5

Employing the obtained temperature distribution onto the structural model, the deflection of the fire affected floor and the column axial forces for the grillage model and the full slab model during the considered fire scenario at floor level 5 are shown in Figures 5.29 and 5.30 respectively, where the variation of the column axial force is again represented by the $P/P_0$ ratio.

With respect to the full slab model, no joint failure is observed at floor level 5 throughout the whole fire. The fire affected column starts to buckle at a time of 25
minutes and 25 seconds (column temperature of 572°C), where due to sufficient load redistribution capacity of the structural system, a stabilized floor mid-span deflection of about 300mm is maintained. The stabilised deflected shape after column buckling of floor level 5 is illustrated in Figure 5.31. As the temperature continues to increase, the overall shear capacity of the four fire affected beam-to-column steel connections decreases to around 18% of the ambient capacity at a time of 30 minutes where the peak temperature is achieved, but no shear failure (punching shear) is observed at the fire affected floor, as illustrated in Figure 5.32. Therefore, sufficient robustness is exhibited of the reference structure subject to fire at floor level 5.

![Deflected shape of floor level 5 at time = 25m25s (full slab model)](image1)

**Fig. 5.31** Deflected shape of floor level 5 at time = 25m25s (full slab model)

![Shear response of fire affected joint for fire at floor level 5 (full slab model)](image2)

**Fig. 5.32** Shear response of fire affected joint for fire at floor level 5 (full slab model)

On the other hand, for the grillage model subject to fire at floor level 5, a poorer performance is observed. Figure 5.33 shows the first joint failure and the successive collapse mode of the grillage model. The first joint failure is observed at a time of 25 minutes 20 seconds in the fire affected minor axis beam-to-column joint under hogging moment, where the elongation of the highest bolt-row exceeds the maximum limit of
35mm. This is a typical ‘single-span’ failure type which is due to insufficient ductility offered by the fire affected joint (subject to hogging moment) supporting a single-span beam when the supported fire affected column still maintains its resistance. Successive joint failure is found in the ambient minor axis beam-to-column joints located at the other end of the secondary beam, and at the same time, the fire affected column starts to buckle. The structure exceeds its robustness limit state due to insufficient ductility supply provided by the surrounding ambient joints very soon after the first joint failure. Clearly, the less robust response obtained by the grillage model compared with the full slab model is mainly due to the neglect of 2D slab effects that can greatly contribute to the vertical resistance of floor systems.

![First and successive joint failures of grillage model for fire at floor level 5](image)

**Fig. 5.33** First and successive joint failures of grillage model for fire at floor level 5

### 5.5.4 Structural robustness – fire at floor level 8

The considered sub-structural model for floor level 8 is only comprised of the fire affected floor system without the spring representing upper ambient floors. This is different from the models of the other two fire cases at floor levels 1 and 5 where the resistance offered by the upper ambient floors can be relied on. The floor deflections and the column axial forces for the grillage model and the full slab model during the considered fire scenario are shown in Figures 5.34 and 5.35 respectively.

For the structural response predicted by the full slab model, first joint failure is observed at a time of 27 minute 10 seconds along with column buckling in the fire affected major axis beam-to-column joint under sagging moment, which is governed by the 25mm limit of the rupture elongation of the lowest bolt-row (furthest from the centre of compression). It is evident that this first joint failure mode is directly caused by the buckling of the column, and subsequently the fire affected floor system deflects significantly to withstand the vertical loading on its own in double span with the
absence of contributions from upper ambient floors. The deflected shape after column buckling of floor level 8 is illustrated in Figure 5.36.

**Fig. 5.34** Deflection of fire affected floor for fire at floor level 8

**Fig. 5.35** Column axial forces of fire affected column for fire at floor level 8

**Fig. 5.36** Deflected shape of floor level 8 at time = 25m25s (full slab model)
Importantly, no shear failure (punching shear) is observed in the fire affected beam-to-column joint before column buckling, and this can be explained through Figure 5.37, where the corresponding transferred shear force and the shear capacity of the fire affected joints are illustrated. Despite a complete loss of the resistance of the fire affected column, which directly leads to the first failure of the fire affected major axis beam-to-column joint, no successive surrounding ambient joint failure is induced in the double-span top floor system bridging over the buckled column. The final floor deflection is arrested at around 750mm without triggering progressive collapse. Therefore, sufficient robustness is exhibited for the reference car park subject to localised fire at the top floor when the full slab model is considered.

![Fig. 5.37 Shear response of fire affected joint for fire at floor level 8 (full slab model)](image)

On the other hand, the grillage model predicts a different first joint failure mode, which occurs before column buckling in the secondary beam where the fire affected minor axis beam-to-column joint fails under hogging moment due to the rupture of the highest bolt-row (the elongation exceeds the maximum limit of 35mm), as shown in Figure 5.38(a). Afterwards, successive failure is induced in the ambient minor axis beam-to-column joints located at the other end of the secondary beam, as shown in Figure 5.38(b). No column buckling is observed due to a relatively lightly load resisted by the top floor fire affected column. It should be noted that the collapse stays localised in the failed secondary beam in a ‘single-span’ manner (i.e. the collapse of the secondary beam only).
This is again attributed to the neglect of the 2D slab effect in the grillage model where the contributions from adjacent parts of the floor slab cannot be relied on.

Fig. 5.38 First and successive joint failures of grillage model for fire at floor level 8

5.5.5 Summary of robustness assessment

Three first joint failure types are generally observed in the reference building subject to the selected fire scenarios, namely, ‘single-span failure’ type, ‘double-span failure’ type and ‘shear failure’ (punching shear) type, as illustrated in Figure 5.39.

Fig. 5.39 Typical fire-induced first joint failure types

The ‘single-span failure’ type usually occurs under the condition where the fire affected column maintains its strength during a fire, or the upper ambient floors can offer sufficient resistance for the fire affected floor system with an acceptably small deflection after the buckling of the column, while the single-span beams are unable to survive due to failure of the supporting joints. This failure type has been found for the considered car park only in the grillage model.

The ‘double-span failure’ type is associated with the case where the fire affected floor and the upper ambient floors do not have sufficient ductility to resist and redistribute gravity load on their own in double span after the buckling of the fire affected column.
This failure type is typical for ‘column loss’ scenarios, and has been found in both full slab and grillage models for the considered car park.

The ‘shear failure’ (punching shear) type is associated with shear failure of steel connections, and it is normally triggered by shear failure of fire affected joints. This failure type can either happen before or after the buckling of the fire affected column, as long as significant shear force is transferred between the column and the connected steel beams. When shear failure (punching shear) occurs, the fire affected floor can be completely detached from the middle supporting and subsequently deflects in a double-span cantilever manner.

Table 5.4 gives the first joint failure time, the first joint failure mode, progressive collapse potential, and the failure type of the reference structure subject to the considered fire scenarios. It is found that the full slab models predict a more satisfactory structural robustness than the grillage model which ignores 2D slab integrity, as illustrated in Figure 5.40. Therefore, it is suggested that although grillage approximations are usually sufficient for conventional structural designs, they tend to be too conservative for structural robustness assessment associated with large deformations, where 2D slab (e.g. membrane action) effects become important.

![Fig. 5.40 Full plot view of grillage and full slab models](image-url)
Table 5.4 Robustness responses of car park under vehicle fire

<table>
<thead>
<tr>
<th>Structural response</th>
<th>Fire floor levels</th>
<th>Full slab model</th>
<th>Grillage model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column buckling time</td>
<td>1</td>
<td>24m56s</td>
<td>25m27s</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>25m25s</td>
<td>26m00s</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>27m10s</td>
<td>No buckling</td>
</tr>
<tr>
<td>First joint failure time</td>
<td>1</td>
<td>30m00s</td>
<td>25m27s</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>No failure</td>
<td>25m20s</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>27m10s</td>
<td>25m20s</td>
</tr>
<tr>
<td>First joint failure position</td>
<td>1</td>
<td>Fire affected beam-to-column steel connections in shear</td>
<td>Fire affected major axis beam-to-column joint under sagging moment</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>No failure</td>
<td>Fire affected minor axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
<td>Fire affected minor axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>First joint failure type</td>
<td>1</td>
<td>Shear failure</td>
<td>Double-span</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>No failure</td>
<td>Single-span</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Double-span</td>
<td>Single-span</td>
</tr>
<tr>
<td>Progressive collapse triggered after first joint failure?</td>
<td>1</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>-</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Successive joint failure position triggering progressive collapse</td>
<td>1</td>
<td>Structure safe</td>
<td>Ambient major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Structure safe</td>
<td>Ambient major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Structure safe</td>
<td>Ambient minor axis beam-to-column joint under hogging moment</td>
</tr>
</tbody>
</table>

5.6 Conclusions

This chapter discusses the response of a reference multi-storey steel-composite car park under a selected localised fire scenario near an internal column, where emphasis is given to realistic robustness assessment of the floor systems subsequent to first joint failure. Based on the proposed fire scenario, actual nonlinear temperature distributions within the structure are adopted, based on detailed thermal analysis performed in SAFIR.
for the project ROBUSTFIRE, and these are subsequently used in ADAPTIC models for structural analysis. Multi-level system models are employed for the structural analysis, where detailed 2D slab models and simplified grillage models of the floor systems have been used to investigate the robustness behaviour of the car park. The definition of overall system failure/progressive collapse is based on comparison of the ductility demand against the ductility supply offered by the surrounding joints, where the component method is employed to characterise the nonlinear joint response. Based on the overall structural analysis, the following comments and outcomes are noted:

- Three types of first joint failure are observed in the reference building subject to the selected fire scenario, namely, single-span failure type, double-span failure type and shear failure type (punching shear). For a typical car park structure where 2D slab effects, e.g. slab membrane action, are considered, the single-span failure type due to the failure of the fire affected joint under normal hogging moment before column buckling is less likely to occur. The other two first joint failure types can be more commonly found, and these are typically associated with column buckling and joint shear failure at elevated temperature.

- Depending on the level of loading, the upper ambient floors can provide an alternative load path for the fire affected floor. Indeed, if the upper ambient floors can resist the redistributed vertical load under a double span configuration with relatively small deflections, and the fire affected floor can resist its load under a single span configuration, no successive collapse is expected unless punching shear occurs, even if the column under fire were to lose its resistance completely due to buckling. Such concepts will be used later in this work to proposed simplified robustness assessment methods for localised fire.

- The grillage model predicts that the reference car park is susceptible to progressive collapse after first joint failure, while the full slab model predicts a more robust structure although first joint failure is also observed. This indicates that grillage approximations which are usually employed for conventional structural designs may be too conservative for structural robustness assessment. Under this circumstance, detailed 2D slab models are recommended for more realistic robustness assessment, particularly for structures where grillage models predict inadequate robustness.
• Based on the structural robustness predicted by the full slab models which indicate low potential for progressive collapse of the reference car park, it can be preliminarily concluded that typical modern multi-storey steel/composite car parks under unfactored gravity load can exhibit favourable robustness even in the absence of additional water sprinkler systems or anti-fire coatings. However, when factored gravity load or other fire locations (near facade columns or corner columns) are to be considered, further investigations are required before the positive outcomes of this study can be generalised.
CHAPTER 6

Simplified Temperature-Dependent Approach (TDA)

6.1 Introduction

A performance-based fire design typically includes two main steps, namely, thermal analysis and structural analysis. With respect to thermal analysis, the most accurate method is to identify the scenario and the scale of the fire, undertake heat transfer analysis, obtain the temperature distribution, and finally apply the temperatures to the structural model, as previously discussed in Chapter 5. However, this approach is often too complicated for design-oriented practice, and does not conform to typical robustness provisions that are intended for unforeseen loading. To avoid complex heat transfer analysis, an alternative approach is developed in this chapter based on a simplified temperature distribution, to provide a reliable yet efficient robustness assessment over the temperature domain instead of the time domain, and is as such event-independent. This idealised temperature distribution is aimed at localised fire scenarios affecting typical multi-storey framed structures.

Of course, the response of structures under idealised event-independent temperature loading should effectively reflect as closely as possible the actual performance of the structure under real localised fires. In other words, only those thermal characteristics which are essential to the robustness assessment of a structure subject to localised fire need to be considered in the idealised temperature distribution. To address this, parametric studies are first conducted in Section 6.2 to investigate the influence of several thermal characteristics on the performance of the reference car park subject to
the actual selected fire scenario proposed in Chapter 5. Based on the findings, a simplified temperature dependent approach (TDA) is developed in Section 6.3. After verifying the reliability of the simplified TDA, illustrative examples are provided to demonstrate its application. Finally, potential dynamic effects associated with column buckling are investigated, where actual double-span floor deflections are compared with those obtained by two idealised extreme cases, namely, ‘static column loss’ and ‘dynamic sudden column loss’.

6.2 Significance of parameters for the TDA

This section investigates the influence of the range of heating zone, the temperature distribution in steel beams, and the temperature distribution in the floor slab on the robustness of the reference car park based on the actual fire scenario. Due to the fact that full slab models can predict a more reliable structural response than grillage models, the behaviour of the full slab model subject to the selected fire scenario presented in Chapter 5 is chosen as the reference case for the current investigation. It is noted that the influence of different temperature parameters on the structural responses to fire at higher floor levels (e.g. top floor) would be more evident than that at lower floors. This is due to the relatively smaller role that the upper ambient floors play when the fire occurs at higher floors, particularly at the top floor where the system response is only determined by the response of the fire affected floor. Therefore, to assess the influence of the different temperature parameters, only the case of fire at the top floor is employed for the current parametric study.

6.2.1 Range of heating zone

According to the thermal analysis results, the high temperature zone is localised to where the structural members of the floor are directly exposed to fire (e.g. ceiling areas immediately above the burning cars) in the vicinity of the fire affected column, while the temperature of the structural members beyond this heated area tends to decrease rapidly. This implies that the temperature distribution outside the localised zone is insignificant towards the robustness assessment of the overall system. This assumption was also proposed in *Handbook 5, Implementation of Eurocodes* (2005), which indicates that for a car park with a typical 5-meter burning car, high temperatures only affect the five meters above the car, while the beam remains at relatively low
temperatures outside the directly-affected region. To investigate the influence of different ranges of heating zone on structural robustness, a reduced heating zone of 10m by 6.7m is considered, where the structural members beyond the reduced heating zone are assumed to be at ambient temperature, as illustrated in Figure 6.1.

Figures 6.2 and 6.3 provide respectively the floor deflection and column axial force of the considered structure with different ranges of heating zone, and several key structural behaviour, including column buckling time, maximum floor deflection after column buckling, first joint failure time, and first joint failure mode, are given in Table 6.1. In general, favourable comparison is observed for the structure affected by the two considered ranges of heating area, although the floor deflection of the structure with the reduced heating area is slightly smaller after column buckling. In addition, both of the cases predict the same first failure mode, which is failure of the fire affected major axis beam-to-column joint under sagging moment. This parametric study indicates that the temperature distribution conditions of the structural members immediately above the burning cars are significant to the robustness response of the multi-storey car park, while the lower temperatures within the adjacent structural members beyond this localised area have limited influence.
CHAPTER 6 Simplified Temperature-Dependent Approach (TDA)

Fig. 6.2 Floor deflection with different heating area

Fig. 6.3 Column axial force with different heating area

Table 6.1 Response of structure subject to different ranges of heating area

<table>
<thead>
<tr>
<th>Response</th>
<th>Real heating area</th>
<th>Reduced heating area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column buckling time</td>
<td>27m10s</td>
<td>27m28s</td>
</tr>
<tr>
<td>Floor deflection after column buckling</td>
<td>725.3mm</td>
<td>689.2mm</td>
</tr>
<tr>
<td>First joint failure time</td>
<td>27m10s</td>
<td>27m28s</td>
</tr>
<tr>
<td>First failure mode/position</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
</tr>
</tbody>
</table>

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6.2.2 Temperature distribution over beam depth

Generally, for a steel beam heated by fire underneath, the temperatures in both the bottom flange and the web can be higher than temperatures in the top flange which contacts the floor slab directly. This thermal gradient over the beam depth can significantly affect the elevated temperature behaviour of an individual steel beam, which is largely due to considerable thermal bowing effects, as pointed out in Chapter 3. From a robustness point of view, however, a double-span type of failure usually dominates the collapse mode. So exceeding the joint ductility supply becomes a critical factor leading to structural failure, whereas the influence of thermal bowing in the steel beams on a double-span system may not be significant. In order to investigate the influence of beam-depth thermal gradients on structural robustness, a simplified uniform distributed temperature field without thermal gradient over the depth of the fire affected steel beams is assumed and compared with the reference case. For the simplified uniform temperature case, the actual beam web temperatures of the reference case are employed.

Figures 6.4 and 6.5 provide respectively the floor deflection and column axial force of the considered structure with the two different steel beam temperature distributions (i.e. real and uniform), and key results for the two cases are given in Table 6.2. It can be seen that the considered structure subject to the assumed uniform temperature distribution over the depth of the steel beam has a similar response to that with real thermal gradients, with a slight delay in column buckling time. This can be explained through Figure 6.5, where a slightly reduced compressive force resisted by the column is shown after a time of 22 minutes for the uniform temperature gradient case, thus leading to longer survival duration of the column before buckling. Based on these findings, it can be preliminary concluded that although neglecting thermal bowing effect tends to postpone buckling and to underestimate the subsequent floor deflections, the assumption of a uniform temperature distribution in the steel beams is reasonable for robustness assessment, provided the fire affected length is not significant compared to the double-span beam length. On the other hand, if the considered fire is extended enough to affect a large part of the double-span system (e.g. fully-engulfed fire), the influence of thermal bowing has to be re-evaluated.
**Fig. 6.4** Floor deflection with different temperature distributions over beam depth

**Fig. 6.5** Column axial force with different temperature distributions over beam depth

**Table 6.2** Response of structure subject to different temperature distributions over beam depth

<table>
<thead>
<tr>
<th>Response</th>
<th>Real beam temperature gradient</th>
<th>Uniform beam temperature gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column buckling time</td>
<td>27m10s</td>
<td>27m40s</td>
</tr>
<tr>
<td>Floor deflection after column buckling</td>
<td>725.3mm</td>
<td>710.9mm</td>
</tr>
<tr>
<td>First joint failure time</td>
<td>27m10s</td>
<td>27m40s</td>
</tr>
<tr>
<td>First failure mode/position</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
</tr>
</tbody>
</table>
6.2.3 Temperature distribution over slab depth

Results from the previous heat transfer analysis indicate that the temperature distribution over the depth of the floor slab is highly non-uniform and complex, though the concrete immediately above the steel deck is found to have a much lower temperature. This enables the possibility of assuming a simplified temperature field over the depth of floor slabs, subject to a rational evaluation of its influence on structural robustness. To address this, a parametric study on temperature distributions over the slab depth is performed, where two other cases are considered and compared with the reference case, namely, *concrete at ambient temperature*, and *entire floor slab at ambient temperature* (i.e. concrete and steel deck). Figures 6.6 and 6.7 provide respectively the floor deflection and the column axial force of the considered structure with the three different slab temperature distributions, and key results are compared in Table 6.3.

Evidently, neglecting the elevated temperature in both concrete and steel deck can considerably underestimate the floor deflections after column buckling, whereas the influence of neglecting the elevated temperature only in concrete is not that significant. This is due to the fact that the steel deck immediately exposed to fire has a much higher temperature than the concrete immediately above. Therefore, significant material degradation in the steel deck can greatly deteriorate the performance of the double-span fire-affected floor bridging over the bucked column, even in the absence of the consideration of the concrete temperature. Another finding is that similar to the phenomenon found in the previous discussion on the steel beam temperature gradients, ignoring the elevated temperature in the concrete or entire slab delays column buckling, which is due to reduced column compressive forces predicted by the two assumed cases, as demonstrated in Figure 6.7. Regarding the failure mode for the case assuming no temperature in the concrete, the same first joint failure mode along with column buckling is found in the fire affected major axis beam-to-column joints under sagging moment at mid-span. On the other hand, the ductility demand is unsafely underestimated when the elevated temperature in both concrete and deck is excluded, where no first joint failure is observed.
CHAPTER 6 Simplified Temperature-Dependent Approach (TDA)

Fig. 6.6 Floor deflection with different temperature distributions over slab depth

Fig. 6.7 Column axial force with different temperature distributions over slab depth

Table 6.3 Response of structure subject to different temperature distributions over slab depth

<table>
<thead>
<tr>
<th>Response</th>
<th>Real slab temperature gradient</th>
<th>Ignore temperature in concrete</th>
<th>Ignore temperature in concrete and deck</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column buckling time</td>
<td>27m10s</td>
<td>28m1ls</td>
<td>29m48s</td>
</tr>
<tr>
<td>Floor deflection after column buckling</td>
<td>725.3mm</td>
<td>686.2mm</td>
<td>422.6mm</td>
</tr>
<tr>
<td>First joint failure time</td>
<td>27m10s</td>
<td>28m1ls</td>
<td>No first joint failure</td>
</tr>
<tr>
<td>First failure mode/position</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
<td>No first joint failure</td>
</tr>
</tbody>
</table>
6.3 Development and verification of simplified TDA

Based on the results from the previous thermal analysis discussed in Chapter 5 as well as the parametric studies, the following basic comments and findings are noted:

1) The high temperature area of a structure subject to a localised fire is only limited within the fire affected column as well as the ceiling immediately above the burning cars, while other parts beyond this localised area are much lower. The elevated temperature in the structural members immediately above the burning cars has a more significant influence on the response of the considered multi-storey car park, while the lower temperatures in the adjacent structural members beyond this localised area have limited influence on structural robustness.

2) Thermal bowing effects induced by the temperature gradients over the depth of the heated steel beams do not significantly affect the response of the double-span floor system in terms of the ductility demand (e.g. floor deflection). Although ignoring the temperature gradient over the beam cross-section can delay column buckling, this effect is rather limited. These findings enable the assumption of uniform temperatures over the steel beam cross-section as a simplified load case during the robustness assessment procedure, provided that the fire and its affected area is localised.

3) The temperatures within the concrete above the heated steel deck are relatively low and have relatively little influence on the overall structural response. Ignoring it can delay column buckling and underestimate the maximum floor deflection, but the effect is quite limited. This finding can be helpful in developing a simplified temperature distribution over the depth of the concrete (e.g. multi-linear approximation) without causing significant inaccuracies, although completely ignoring the concrete temperature also seems to be a simple yet feasible option. On the other hand, high temperatures within the heated steel deck can greatly reduce the resistance of the double-span floor after buckling of the fire-affected column, thus the steel deck temperature has a significant influence on overall ductility demand and hence structural robustness.
Based on these findings, an idealised temperature field within the structural model is developed for the simplified TDA, which allows the elevated temperature structural analysis to be performed in a simplified performance-based manner over the temperature domain instead of the conventional time domain. Therefore, the simplified TDA is relatively event-independent, as it does not require details of fire hazard (e.g. the type, the number or sequence of burning cars) but only requires the range of the fire-affected area. Employing the failure criteria defined in Chapter 5, the maximum temperature that a structure can resist against progressive collapse can then be obtained. The proposed assumptions for the idealised temperature distribution of the reference structure subject to typical localised fires are illustrated in Figure 6.8.

For this simplified temperature distribution, the fire affected area is defined as the area within the range of the burning cars (typically a 7mx10m rectangular area for four burning cars). Due to the high heat conductivity of steel, the temperature in the column is considered to be uniform along the height as well as over the cross-section. The temperatures in the steel beams and the steel decks within the range of burning car areas are considered to be uniform, and the steel beams and the steel decks beyond this fire affected area are assumed to be at ambient temperature. The temperature increase rate for the steel beams and deck is assumed to be the same as that for the column. For concrete, it is assumed that the temperature of the concrete immediately above the steel deck is half of that in the steel deck (which is generally true in the heating phase as observed from the detailed thermal analysis), and it decreases to ambient temperature towards the top face of the slab in a multi-linear manner, as illustrated in Figure 6.8.

Applying the simplified temperature distribution to the structural model of the reference
car park, the floor deflection and column axial force with monotonically increasing temperatures are shown in Figures 6.9 and 6.10, respectively. Figure 6.11 gives the shear response of the fire affected joints, where punching shear is deemed to occur when the total shear force transferred by the connections exceeds their overall shear capacity, and it is simulated by abruptly removing the resistance of fire affected connections completely.

Fig. 6.9 Floor deflection – simplified TDA analysis

Fig. 6.10 Column axial force – simplified TDA analysis
Table 6.4 gives the column buckling temperature, the floor deflection after column buckling, the first failure temperature, and the first failure mode for the cases of fire at floor levels 1, 5, and 8 obtained from the real fire analysis (detailed TDA analysis) and simplified TDA analysis. It can be observed that the simplified TDA analysis predicts a similar structural response to that obtained by the real fire analysis. Although the structural performance predicted by the simplified TDA analysis is slightly less conservative (e.g. predict a slight higher critical temperature), all the discrepancies are within a limited and acceptable range. So as a general remark, the simplified TDA analysis can provide a sufficiently accurate approach to evaluate the structural response under localised fires.

Finally, it is noted that the following idealisations are responsible for the differences between the results obtained by the simplified TDA analysis and the detailed TDA analysis: 1) neglect of temperature in the floor beyond the reduced fire affected area, 2) assumption of uniformly distributed temperature within the beams and decks, 3) assumption of identical temperatures in the beam and deck, 4) idealised temperature distribution in concrete, and 5) neglect of unsymmetrical fire load on different sides of column due to different burning times of affected cars. Notwithstanding, these effects are shown to be relatively unimportant when the structure is considered for robustness assessment.
Table 6.4 Comparison between detailed TDA and simplified TDA analysis

<table>
<thead>
<tr>
<th>Response</th>
<th>Fire affected floor levels</th>
<th>Real fire analysis (peak temperature = 741°C)</th>
<th>Simplified TDA analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column buckling temperature (time)</td>
<td>1</td>
<td>556.1°C (24m56s)</td>
<td>562.5°C</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>580.9°C (25m25s)</td>
<td>590.0°C</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>666.8°C (27m10s)</td>
<td>692.5°C</td>
</tr>
<tr>
<td>Floor deflection after column buckling</td>
<td>1</td>
<td>299.6mm</td>
<td>300.6mm</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>299.9mm</td>
<td>308.9mm</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>725.3mm</td>
<td>683.5mm</td>
</tr>
<tr>
<td>First joint failure temperature (time)</td>
<td>1</td>
<td>741.0°C (30m00s)</td>
<td>780.0°C</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>No first joint failure</td>
<td>825.0°C</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>666.8°C (27m10s)</td>
<td>692.5°C</td>
</tr>
<tr>
<td>First joint failure position/mode</td>
<td>1</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>No first joint failure</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
<td>Fire affected major axis beam-to-column joints under sagging moment</td>
</tr>
<tr>
<td>Progressive collapse triggered after first joint failure?</td>
<td>1</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>-</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

6.4 Application of simplified TDA

The robustness assessment procedure employing the simplified TDA is mainly comprised of structural analysis followed by the evaluation of ductility capacity-demand ratio (CDR). To illustrate this procedure in more detail, the reference car park discussed in Chapter 5 is again employed in this section, where the 2D full slab modelling approach is selected due to its reliability. For each considered floor level (levels 1, 5, and 8) under fire, different values of vertical gravity loading are considered for comparison with the case considering the unfactored basic load of 5kN/m² (2.5kN/m² dead load + 2.5kN/m² live load), namely, 4kN/m², 6kN/m² and 7kN/m². It is noted that a slight change in the characteristics of the column top vertical spring
CHAPTER 6 Simplified Temperature-Dependent Approach (TDA)

representing the upper ambient floors may be required when different vertical loadings are employed, and this is also considered in the current structural model.

6.4.1 Fire at floor level 1

The structural model for the case of fire at the ground floor is comprised of one fire affected floor system and seven upper ambient floor systems for which the characteristics (e.g. stiffness and ductility supply) are incorporated into a nonlinear spring at the top of the fire affected floor model, as already elaborated in Chapter 5. S460 HEB550 steel column is employed as the internal column exposed to localised fire, and the simplified TDA temperature distribution is applied locally onto the internal column as well as the adjacent floor system. Figures 6.12 and 6.13 present the floor deflection and column axial force respectively for fire at floor level 1 under the different gravity loads, noting that the initial column axial force $P_0$ is different between the depicted UDL cases. Considering possible shear failure of the fire-affected steel connections, which leads to punching shear, Figure 6.14 provides the transferred shear forces of floor level 1 for predicting the corresponding punching shear temperatures.

![Fig. 6.12 Floor deflection of for fire at floor level 1 – simplified TDA](image)

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In general, no progressive collapse is triggered up to a gravity UDL of 6kN/m² up to the maximum temperature of 1000°C, even though first joint failure is observed due to shear failure (punching shear). Under the same fire conditions, the structure is deemed to have a high potential for progressive collapse when an increased UDL of 7kN/m² is considered, where progressive collapse is governed by the successive failure of the surrounding ambient joint due to rebar rupture immediately after punching shear.
Before column buckling, no local failure is found in the joints directly exposed to fire under a gravity load up to 7kN/m², which again confirms that ‘single-span type’ collapse is less likely to occur in full slab models. As the temperature keeps increasing, the fire affected column starts to buckle. It is evident from Figure 6.13 that the localised fire leads to a complete loss of column resistance immediately after buckling under the gravity loads of 5kN/m², 6kN/m², and 7kN/m², while under the reduced vertical UDL of 4kN/m², the column buckling procedure is more gradual. For all the four load cases, no immediate overall system failure is observed along with column buckling, which is attributed to the stronger upper ambient floors that offer additional resistance for the fire affected floor, and subsequently the vertical floor deflections are thus arrested at around 150 mm, 300mm, 430mm, and 540mm under the UDLs of 4kN/m², 5kN/m², 6kN/m² and 7kN/m² respectively. Once the vertical floor deflections start to stabilise after column buckling, no gradual increase of deflection is observed with temperature due to the resistance provided by the upper ambient floors.

The stabilised deflection of the fire affected floor after column buckling is disturbed when higher temperatures are achieved. This is due to the shear failure (punching shear) of the fire affected beam-to-column steel joint, which occurs under all the four considered gravity loadings, as shown in Figure 6.14. When the overall shear resistance of the joint (combining the four steel connections) at elevated temperature drops below the overall transferred shear force, first joint failure of the system is triggered by means of punching shear. Upon punching shear, the fire affected floor is assumed to be completely detached from the upper ambient floors from the location of the fire affected joint. It is observed in the current model that punching shear typically occurs at temperatures of around 250°C to 350°C higher than the column buckling temperature.

To evaluate the progressive collapse potential of the fire affected floor after punching shear, Figure 6.15 provides the rebar elongations of the surrounding ambient major axis composite joints of the fire affected floor, where rebar rupture is deemed to govern the failure mode of the joints under hogging moments and thus to limit the ductility supply of the detached fire affected floor. It is observed that under the UDLs of 4 - 6kN/m², the rebar elongations remain below the rupture value (predicted by Anderson et al., 2000) throughout the entire heating procedure, thus further deflections of the detached fire affected floor after punching shear are safely arrested at approximately 500mm, 600mm,
and 700mm, respectively, due to sufficient ductility supply provided by the surrounding ambient joints. On the other hand, punching shear directly leads to successive rebar rupture of the surrounding ambient joints when an increased UDL of 7kN/m² is considered, which triggers collapse of the detached fire affected floor.

Fig. 6.15 Rebar elongation in surrounding joints for fire at floor level 1—simplified TDA

6.4.2 Fire at floor level 5

With respect to the case of fire at floor level 5, the structural model is comprised of the fire affected floor and the three upper ambient floor systems where the characteristics are lumped into a nonlinear spring. S460 HEB300 steel column is employed as the internal column exposed to localised fire. Figures 6.16 and 6.17 provide the floor deflections and axial column forces for the floor under the four different gravity loads. Similar to the structural response observed in the case of fire affecting floor level 1, no progressive collapse is triggered under gravity UDL of up to 6kN/m², where column buckling leads to no immediate first joint failure, although the compressive resistance of the column is completely lost. Evidently, this is due to the additional resistance provided by the three upper ambient floors, and therefore the vertical floor deflections are safely arrested at around 165 mm, 300mm, and 420mm under the UDLs of 4kN/m², 5kN/m², and 6kN/m² respectively. First joint failure is initiated by means of punching shear when the shear resistance of the fire affected joint drops below the transferred shear force. Figure 6.18 shows the transferred shear force around the internal column for the fire-affected floor. When punching shear occurs, the fire-affected floor is
detached from the upper ambient floors, and the final deflections of the detached fire floor are arrested at approximately 500mm, 580mm, and 680mm under the UDLs of 4kN/m², 5kN/m², and 6kN/m², respectively. Nevertheless, with sufficient ductility supply provided by the surrounding ambient joints of the fire affected floor, no successive joint failure occurs, thus no progressive collapse is triggered.

Fig. 6.16 Floor deflection for fire at floor level 5 – simplified TDA

Fig. 6.17 Column axial force for fire at floor level 5 – simplified TDA
Under an increased UDL of 7kN/m², however, the fire affected column starts to buckle at a temperature of 500°C, which directly leads to rebar rupture of the surrounding ambient joints, as shown in Figure 6.19. According to the predefined structural failure criteria, this joint failure position can directly cause progressive collapse of the overall structure. This is different from the response observed for fire affecting floor 1, where progressive collapse is initiated under the gravity loading of 7kN/m² after column buckling by punching shear.
6.4.3 Fire at floor level 8

For the top floor, no above ambient floors can be relied on, hence the structural model is only comprised of one fire affected floor system with a HEB 220 steel column. Therefore, different failure responses may be expected compared with the other two cases considering fires at lower floor levels. Under the simplified TDA temperature distribution, the floor deflections and axial column forces with increasing temperature are shown in Figures 6.20 and 6.21, respectively.

It can be seen that at a relatively light gravity load of 4kN/m$^2$ or 5kN/m$^2$, no progressive collapse is triggered up to the maximum considered temperature of 1000$^\circ$C, even though first joint failure is detected in the fire affected major axis joint under sagging moment along with column buckling, where the elongation of the lowest bolt-row (furthest from the centre of compression) exceeds the limit of 25mm. It is evident from Figure 6.21 that the localised fire at the top floor leads to a complete loss of column resistance due to buckling. However, progressive collapse is prevented due to sufficient ductility supply provided by the surrounding ambient joints, and the deflections of the top floor after column buckling are thus arrested under these two levels of gravity loading at 500mm and 700mm, respectively, in a double-span manner. After column buckling, a gradually increased deflection is observed until the maximum considered temperature of 1000$^\circ$C, which is caused by further reduction of floor resistance with temperature. The behaviour of the top floor as well as the performance of the detached fire affected floor discussed in the previous cases for fire at floor levels 1 and 5 shows that local failure of the fire affected joints, either under sagging bending moment or subject to shear failure (causing punching shear), does not necessarily initiate progressive collapse. This implies that from a robustness perspective, the benefits of the 2D slab effect (e.g. membrane action) and the ductility supply offered by the surrounding ambient joints can be significant in mitigating progressive collapse.
On the other hand, considering the cases of larger UDL’s (6kN/m² and 7kN/m²), progressive collapse is triggered along with column buckling immediately after first joint failure of the fire-affected major axis beam-to-column joint under sagging moment. The successive joint failure that potentially induces progressive collapse is found in the surrounding ambient major axis beam-to-column joint where the rebars rupture. The corresponding rebar elongations of the surrounding ambient major axis joints are shown in Figure 6.22.
Importantly, no shear failure (punching shear) is observed in the fire affected beam-to-column steel connections up to 1000°C under the four considered gravity loads. This can be explained through Figure 6.23, where the corresponding transferred shear force and the overall shear capacity of the fire affected joint are illustrated. It is found that before column buckling, no premature shear failure occurs, although column buckling at the top floor is significantly delayed compared to the cases of fire affecting the other two lower floor levels. When the column buckles, the floor system tends to deflect in a pure double-span manner in the absence of upper ambient floors, hence the shear force
transferred by the fire affected steel joint in the mid-span immediately decreases to zero. Therefore, punching shear is not expected at the top floor if no joint shear failure is observed before column buckling.

6.4.4 Summary of simplified TDA analysis

Summarising the structural behaviour of the reference building subject to fire at floor levels 1, 5 and 8 under the simplified TDA analysis, it can be observed that progressive collapse is triggered for all the three considered floors under a gravity load of 7kN/m². Under a gravity load of 6kN/m², the potential for progressive collapse is only high for fire at the top floor. No progressive collapse is observed for all the considered fire cases when a gravity load of less than 5kN/m² is considered. Importantly, this study also sheds light on deficiencies of the conventional codified treatments on structural fire design. For the conventional fire design, emphasis is usually given to the fire resistance of individual members. Considering a fire affected column, the failure criterion for the system is typically defined at the moment when the column axial force reduces back to its original ambient value (Franssen, 2000; Wang, 2004; Huang and Tan, 2004). According to this failure criterion, all cases considered in the current study should have experienced failure since all the corresponding columns have completely lost their axial compressive resistance below the maximum temperature of 1000°C. Indeed, most of the cases do not lead to floor collapse, which clearly suggests that the conventional fire design procedure focusing on individual member resistance is too conservative.

Figure 6.24 summarises the column buckling temperatures of all the considered cases. It is clear that the column buckling temperature tends to decrease with gravity loading, and is largely dependent on the initial load ratio of the column. For fires at floor levels 1 and 5, where the initial load ratios are relatively high, the buckling temperatures are on average 100°C to 150°C lower than those of the top floor column which is more lightly loaded. Importantly, all the columns buckle below a temperature of 750°C which is typically attained under a medium to severe localised fire in an open car park. Therefore, from a robustness point of view, considering a column loss scenario during a localised fire in the absence of knowledge of the maximum temperature is rational and conservative. However, the question remains whether to consider static column loss or to consider potential dynamic effects; this issue will be addressed in Section 6.5.
Considering the floor deflections/ductility demands, Figure 6.25 provides the deflections of the fire-affected floors at levels 1, 5, and 8 immediately after column buckling as well as the floor deflections after the occurrence of punching shear for fire at floor levels 1 and 5. It is clear that the deflections of the fire affected floor of levels 1 and 5 immediately after column buckling are much smaller than those of the top floor where no contribution from upper ambient floors can be relied on. This confirms that when upper ambient floors have sufficient resistance to withstand gravity load in double-span on their own, and also offer additional resistance to share the load of the fire-affected floor, the ductility demand in the overall system can be greatly reduced compared with the individual fire affected floor withstanding the gravity load on its own. Therefore, for a typical steel/composite building such as the reference car park considered here, the potential for progressive collapse due to column loss for fire at lower floors is evidently less than that at higher floors, provided punching shear failure is avoided.

After punching shear, however, the deflections of the detached fire affected floors for fire at floor levels 1 and 5 are close to the top floor deflections, as shown in Figure 6.25. This indicates that when the temperature is sufficiently high to cause punching shear for fire at lower floor levels, the final deflection for fire at any floor can be similar. Of course, differences can still be found due to different failure modes (i.e. column
buckling for fire at the top floor and punching for fire at lower floors) which may induce different levels of dynamic effects.

Fig. 6.25 Summary of floor deflections – simplified TDA

Finally, the failure characteristics of the reference building using the simplified TDA analysis are summarised in Table 6.5. It is observed that punching shear tends to govern the first joint failure mode of the structure subject to fire at lower floor levels, but this normally occurs at a rather late stage (well beyond column buckling) under a severe fire where a sufficiently high joint temperature is developed. Even if punching shear occurs, successive progressive collapse can still be prevented if the gravity load is not excessive. On the other hand, typical ‘double-span’ first joint failure type tends to occur for fire at the top floor. This first failure type is normally associated with column buckling, and is more likely to initiate successive joint failure at higher floor levels where limited contributions from the upper ambient floors is available subsequent to column loss. In this context, under an intermediate fire accident which does not lead to punching shear, more precaution should be undertaken to prevent progressive collapse for fire at higher floor levels than at lower floor levels, particularly when ambient floors can have sufficient resistance/ductility to accommodate their double-span deflection after column loss.
### Table 6.5 Structural failure under fire – simplified TDA analysis

<table>
<thead>
<tr>
<th>Structural failure response</th>
<th>UDL</th>
<th>Fire at floor level 1</th>
<th>Fire at floor level 5</th>
<th>Fire at floor level 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>First joint failure temperature</td>
<td>4kN/m²</td>
<td>825°C</td>
<td>840°C</td>
<td>725°C</td>
</tr>
<tr>
<td></td>
<td>5kN/m²</td>
<td>780°C</td>
<td>825°C</td>
<td>690°C</td>
</tr>
<tr>
<td></td>
<td>6kN/m²</td>
<td>785°C</td>
<td>805°C</td>
<td>667.5°C</td>
</tr>
<tr>
<td></td>
<td>7kN/m²</td>
<td>765°C</td>
<td>800°C</td>
<td>645°C</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>First joint failure position</th>
<th>4kN/m²</th>
<th>Fire affected beam-to-column steel connections in shear (punching shear)</th>
<th>Fire affected beam-to-column steel connections in shear (punching shear)</th>
<th>Fire affected major axis beam-to-column joint under sagging moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5kN/m³</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
<td>Fire affected major axis beam-to-column joint under sagging moment</td>
</tr>
<tr>
<td></td>
<td>6kN/m²</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
<td>Fire affected major axis beam-to-column joint under sagging moment</td>
</tr>
<tr>
<td></td>
<td>7kN/m²</td>
<td>Fire affected beam-to-column steel connections in shear (punching shear)</td>
<td>Surrounding ambient major axis beam-to-column joint under hogging moment</td>
<td>Fire affected major axis beam-to-column joint under sagging moment</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Progressive collapse triggered after first joint failure?</th>
<th>4kN/m²</th>
<th>No</th>
<th>No</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5kN/m²</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>6kN/m²</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>7kN/m²</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Successive joint failure position triggering progressive collapse</th>
<th>4kN/m²</th>
<th>Structure safe</th>
<th>Structure safe</th>
<th>Structure safe</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5kN/m²</td>
<td>Structure safe</td>
<td>Structure safe</td>
<td>Structure safe</td>
</tr>
<tr>
<td></td>
<td>6kN/m²</td>
<td>Structure safe</td>
<td>Structure safe</td>
<td>Structure safe</td>
</tr>
<tr>
<td></td>
<td>7kN/m²</td>
<td>Surrounding ambient major axis beam-to-column joint under hogging moment</td>
<td>Surrounding ambient major axis beam-to-column joint under hogging moment</td>
<td>Surrounding ambient major axis beam-to-column joint under hogging moment</td>
</tr>
</tbody>
</table>

#### 6.4.5 Evaluation of Capacity-Demand Ratio (CDR)

From a robustness perspective, a capacity/demand ratio (CDR) is usually used to indicate the proximity of the structure to a limit state, which is typically expressed in terms of the associated structural resistance compared to the applied loading, where a
ratio exceeding a value of 1.0 indicates a safe structure. Clearly, under localised fire conditions, the capacity/demand ratio for a system depends on temperature. In other words, whether a structure can survive under fire depends on the severity of the fire (or the maximum temperature that can be reached). To address this, three peak temperatures are considered in this study for CDR assessment, namely, 500°C, 750°C and 1000°C. These three maximum temperatures represent respectively three typical different severities or probabilities of expected fire, i.e. ‘frequent/basic’, ‘intermediate’, and ‘rare’. This classification strategy is similar to that used in seismic engineering, where different earthquake occurrence frequencies are employed to deduce different levels of seismic loadings applied on structures. Towards future codification, this strategy can potentially enable a reasonable robustness CDR assessment procedure through applying different pre-determined expected peak temperatures on structures with various functions and significance. Table 6.6 provides the CDRs for the current structure considering a basic UDL of 5kN/m², and the CDRs are found to correlate well with the previously discussed structural response.

<table>
<thead>
<tr>
<th>Fire floor level</th>
<th>Maximum temperature expected (°C)</th>
<th>UDL capacity (kN/m²)</th>
<th>CDR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>frequent (500)</td>
<td>7.00</td>
<td>1.400</td>
</tr>
<tr>
<td></td>
<td>Intermediate (750)</td>
<td>7.00</td>
<td>1.400</td>
</tr>
<tr>
<td></td>
<td>Rare (1000)</td>
<td>6.59</td>
<td>1.318</td>
</tr>
<tr>
<td>5</td>
<td>frequent (500)</td>
<td>7.00</td>
<td>1.400</td>
</tr>
<tr>
<td></td>
<td>Intermediate (750)</td>
<td>6.95</td>
<td>1.390</td>
</tr>
<tr>
<td></td>
<td>Rare (1000)</td>
<td>6.76</td>
<td>1.352</td>
</tr>
<tr>
<td>8</td>
<td>frequent (500)</td>
<td>12.80</td>
<td>2.560</td>
</tr>
<tr>
<td></td>
<td>Intermediate (750)</td>
<td>5.67</td>
<td>1.134</td>
</tr>
<tr>
<td></td>
<td>Rare (1000)</td>
<td>5.67</td>
<td>1.134</td>
</tr>
</tbody>
</table>

It is shown that the CDRs exceed 1.0 for all cases, thus indicating sufficient robustness of the reference structure for the basic gravity load of 5kN/m². For the case of floor levels 1 and 5 under fire, the CDR tends to decrease with temperature but is always larger than 1.20, which confirms that the structure indeed has sufficient robustness even for UDL exceeding 6kN/m². The CDR for fire at the top floor generally drops with
temperature, but tends to achieve a residual value at large temperature. This residual value exceeds 1.0, which confirms that a UDL of 5kN/m² can indeed be sustained by the top floor at very high temperature without overall system failure. The CDR for the top floor at a peak temperature of 500°C is very high (2.560), and this is due to the fact that the top floor column is relatively lightly loaded, where the column buckling temperature under typical gravity loadings significantly exceeds the considered peak temperature of 500°C. When an intermediate or a rare fire accident is considered, the CDRs for the top floor drop rapidly below 1.20. In this case, progressive collapse may be expected for the top floor under a gravity load of 6kN/m². For the current top floor model, the CDRs under intermediate and rare fires are identical, which implies that due to the definition of structural failure based on the failure of the surrounding ambient joints, the temperature within the fire affected area has negligible influence on the overall structural robustness after column loss for fire at the top floor.

Comparing the structural fire response at different floor levels under a frequent fire (maximum temperature of 500°C), the CDR for the top floor is much higher than that at the other floor levels where the columns are more prone to buckle under a peak temperature of 500°C. At a higher maximum temperature of 750°C (before punching shear), where the columns at all the considered floors are completely buckled under the considered gravity loading, the CDRs of floor levels 1, 5, and 8 are 1.400, 1.390, and 1.134 respectively, which follow a decreasing trend. This again confirms the importance of additional resistance offered by upper ambient floors. Under a maximum temperature of 1000°C (after punching shear), a difference of CDR between floor level 1/5 (1.318 and 1.352) and floor level 8 (1.134) is still observed, although the final deflecting mechanisms for the three cases are similar (i.e. individual fire affected floor resists gravity load in double-span on its own). This is mainly because of the two different failure modes (punching shear and column buckling) that induce different final deflections of the fire affected floor. Punching shear occurs at a deflection of 300mm after column buckling, which tends to induce less dynamic effects, hence floor deflection/ductility demand, than that at the top floor where the fire affected floor start to deflect in double-span from approximately zero deflection initiated by column buckling. Therefore, due to the reduced dynamic effects for fire affecting floor levels 1 and 5, the corresponding CDRs are larger than for the case of fire affecting the top floor.
6.5 Significance of dynamic effects

It is now widely recognised that dynamic effects associated with idealised sudden column loss scenarios play an important role in structural robustness assessment procedures considering blast loading (Izzuddin et al., 2008). Considering fire conditions where columns are directly exposed to fire, potential dynamic effects are also potentially likely due to column buckling. However, conventional research regarding the structural behaviour of multi-storey steel/composite car parks subsequent to column loss due to burning cars normally assumes a static response (Crosti et al., 2009; Zanon et al., 2010), which could unsafely underestimate the ductility demand.

As observed from the vertical deflections of the reference structure subject to monotonically increasing temperatures in the TDA, dynamic effects feature in most cases by the evident vibrations of vertical deflection after column buckling. Indeed, when a fire affected column buckles, the scale of dynamic effects is largely associated with the variation of residual column resistance with top vertical displacement. Dynamic effects can be greatly reduced or even eliminated if the column possesses sufficient residual resistance at significant top displacement after buckling. On the other hand, when the buckled column tends to lose most of its resistance at the onset of top vertical displacement, significant dynamic effects can be triggered, thus leading to an increased ductility demand.

While the column resistance-deformation characteristics can be used to explain dynamic effects, these can alternatively be viewed in terms of ‘buckling duration time’, which is the time interval from when the column axial resistance starts to drop with incremental mechanical displacements for a fixed temperature to when a considerable proportion of this resistance has been lost. If the buckling duration is very short compared to the time it takes the full system to stabilise, which corresponds to a sharp reduction in column resistance with mechanical displacement, the subsequent system response is very similar to one obtained assuming sudden column loss. On the other hand, for a relatively long buckling duration, the column loss can be considered to be static.

In order to evaluate the scale of dynamic effects associated with column buckling due to fire, this study considers two additional idealised extreme conditions, namely, static column buckling process, and sudden dynamic column loss, as illustrated in Figure 6.26.
CHAPTER 6 Simplified Temperature-Dependent Approach (TDA)

For static column buckling, the vertical resistance provided by the fire affected column is assumed to decrease gradually at the buckling temperature in order to eliminate dynamic effects completely. On the other hand, for sudden column loss, the vertical resistance of the column at the buckling temperature is assumed to be lost abruptly over a very short time of 1 msec, and a detailed dynamic analysis is performed to obtain the deflections. The actual floor deflections after column buckling are expected to fall between the two idealised extreme conditions. For the two extreme situations, the vertical resistance offered by the fire-affected column before buckling is reproduced through employing a varying upwards point load identical to the actual compressive force resisted by the column. Through this strategy similar behaviour of the floor system before column buckling can be ensured, which can then followed by different assumptions regarding the nature of column buckling, i.e. static, actual, or sudden.

![Diagram showing different column loss strategies under fire](image)

Fig. 6.26 Illustration of different column loss strategies under fire

Through incorporating the three values of final deflection after column loss, a new simple index, which is named as the Dynamic Effect Factor (DEF), is proposed here to conveniently quantify the significance of dynamic effects during the buckling of the fire affected column, as expressed in the following equation:

$$ DEF = \frac{(D_a - D_s)}{(D_d - D_s)} $$  \hspace{1cm} (6.1)
where $D_a$ is the actual floor deflection subsequent to column buckling, $D_s$ is the static floor deflection after column buckling, and $D_d$ is the dynamic floor deflection after sudden column loss. The reason for selecting deflections as the dynamic evaluation benchmark for the index is obviously because that structural ductility demand is closely associated with the maximum double-span floor deflection, where the critical value is governed by failure of the surrounding ambient joints. The value of $DEF$ can range from 0.0 to 1.0 indicating a static column loss process and a sudden column loss process, respectively. Figures 6.27 to 6.29 show the three deflections (i.e. actual, static, and sudden column loss) of the reference building subject to fires at floor levels 1, 5, and 8, respectively. The corresponding values of deflection and $DEF$ are listed in Tables 6.7 to 6.9, where the dynamic deflections under actual and sudden column loss are taken as the first maximum values after column loss.

As observed from the results, dynamic effects associated with column buckling are featured for most cases, and they tend to be more predominant with increasing gravity load. For fire at floor levels 1 and 5, the $DEF$ is zero when a reduced UDL of 4kN/m$^2$ is considered, thus indicating a static response of the floor system along with column buckling. As the gravity load increases, the $DEF$ starts to increase towards a more evident dynamic response, where the maximum value is 0.623 under a UDL of 7kN/m$^2$. More significant dynamic effects are observed when fire occurs at the top floor where the $DEF$ ranges from 0.691 to 0.864 under the gravity loads from 4kN/m$^2$ to 7kN/m$^2$. From the obtained $DEF$, it can be concluded that conventional static analysis for the structural response to column buckling under fire is unsafe, while assuming sudden column loss can be too conservative. Therefore, performing a dynamic analysis, for example using the simplified TDA, can predict a more realistic ductility demand.
Fig. 6.27 Floor deflections subject to various column loss scenarios – fire at floor level 1

Table 6.7 Dynamic effects assessment – fire at floor level 1

<table>
<thead>
<tr>
<th>UDL</th>
<th>Full static deflection $D_s$ (mm)</th>
<th>Actual deflection $D_a$ (mm)</th>
<th>Sudden column loss deflection $D_d$ (mm)</th>
<th>Dynamic Effect Factor (DEF)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>4kN/m²</td>
<td>150.2</td>
<td>150.2</td>
<td>307.0</td>
<td>0.000</td>
</tr>
<tr>
<td>5kN/m²</td>
<td>217.0</td>
<td>309.2</td>
<td>423.9</td>
<td>0.446</td>
</tr>
<tr>
<td>6kN/m²</td>
<td>283.7</td>
<td>437.9</td>
<td>553.5</td>
<td>0.572</td>
</tr>
<tr>
<td>7kN/m²</td>
<td>349.5</td>
<td>550.8</td>
<td>680.1</td>
<td>0.609</td>
</tr>
</tbody>
</table>

*value ranges from 0 (static response) to 1 (sudden column loss response)
**Fig. 6.28** Floor deflections subject to various column loss scenarios – fire at floor level 5

**Table 6.8** Dynamic effects assessment – fire at floor level 5

<table>
<thead>
<tr>
<th>UDL (kN/m²)</th>
<th>Full static deflection (D_s) (mm)</th>
<th>Actual deflection (D_a) (mm)</th>
<th>Sudden column loss deflection (D_d) (mm)</th>
<th>Dynamic Effect Factor (DEF)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>165.6</td>
<td>165.6</td>
<td>322.9</td>
<td>0.000</td>
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<tr>
<td>5</td>
<td>238.4</td>
<td>308.9</td>
<td>462.3</td>
<td>0.315</td>
</tr>
<tr>
<td>6</td>
<td>305.1</td>
<td>426.1</td>
<td>587.8</td>
<td>0.428</td>
</tr>
<tr>
<td>7</td>
<td>370.4</td>
<td>579.9</td>
<td>706.7</td>
<td>0.623</td>
</tr>
</tbody>
</table>

*value ranges from 0 (static response) to 1 (sudden column loss response)
Fig. 6.29 Floor deflections subject to various column loss scenarios – fire at floor level 8

Table 6.9 Dynamic effects assessment – fire at floor level 8

<table>
<thead>
<tr>
<th>UDL</th>
<th>Full static deflection $D_s$ (mm)</th>
<th>Actual deflection $D_a$ (mm)</th>
<th>Sudden column loss deflection $D_d$ (mm)</th>
<th>Dynamic Effect Factor ($DEF^*$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4kN/m²</td>
<td>284.3</td>
<td>502.5</td>
<td>599.9</td>
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<tr>
<td>5kN/m²</td>
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<td>6kN/m²</td>
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<td>7kN/m²</td>
<td>502.4</td>
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*value ranges from 0 (static response) to 1 (sudden column loss response)
Finally, it is worth noting that conducting a dynamic TDA analysis on structures/substructures may be too complex for robustness assessment in design practice. In this respect, if the column axial response under its buckling temperature is known, it is possible to develop a simplified method to estimate the dynamic effects. A new simplified method based on this hypothesis will be developed in Chapter 8.

### 6.6 Conclusions

An event-insensitive robustness assessment approach, the simplified TDA, is developed in this chapter, where an idealised monotonically increasing temperature distribution is proposed. The simplified TDA is verified to have sufficient accuracy compared with the predictions obtained by a more detailed TDA employing actual nonlinear temperature distributions. A novel feature of the simplified TDA is its capability of evaluating the capacity-demand ratio of a structure subject to localised fire based on predefined failure criteria provided the expected maximum temperature can be established. Different severity of fire can lead to different temperature dependent capacity-demand ratios. Compared with the detailed TDA, the simplified TDA conforms more closely to typical robustness provisions, which consider the mitigation of progressive collapse under event-insensitive conditions. From the presented case study using the simplified TDA, the following conclusions and comments are noted:

- The maximum temperature leading to progressive collapse depends on a number of factors, including gravity load, column size, location/floor level of fire, and the number of ambient floors above the fire affected floor. In general, the potential for progressive collapse directly caused by column buckling for fire at lower floor levels is lower than that for fire at higher floor levels, provided that ambient floors have favourable double span resistance. When a more severe fire is considered which eventually leads to punching shear, the difference with regard to progressive collapse potentials for fire at lower floors and higher floors is narrowed due to a similar final floor resisting mechanism (i.e. individual fire-affected floor resists gravity load in double span).

- Overall system failure can occur at temperatures that are much greater than the conventional definition of ‘critical temperature’ of the fire affected column based on the axial force reducing back to its ambient value. In the current
reference building, evident residual column resistance beyond the ‘critical temperature’ is observed for fire at floor level 1 or 5 under a gravity load of 4kN/m\(^2\). When a larger gravity load is considered, however, the residual column resistance is significantly reduced immediately after the ‘critical temperature’. Even so, progressive collapse can still be avoided for a gravity load less than 6kN/m\(^2\) due to sufficient resistance of the double-span floor systems.

- The main mode that potentially triggers progressive collapse is found in the reference building to be governed by the rupture of the rebars in the surrounding ambient composite joints under hogging moment. While it is possible for a structure to survive after the rupture of slab rebars through utilising the residual capacity of the bare steel joints in conjunction with 2D slab effects, such a mechanism has not been fully studied and thus needs further investigation and experimental validation. Therefore, the current failure prediction on the structural system level is considered to be on the conservative side.

- Dynamic effects arise for the structural system due to column buckling under fire. The corresponding final floor deflection is found to fall between two idealised cases, which are the static column loss and sudden column loss scenarios. It is also observed that more dynamic effects arise when localised fire occurs at the top floor. In addition, a larger gravity load can lead to more significant dynamic effects. In order to predict a reliable ductility demand of a car park after column loss due to localised fire, the consideration of dynamic effects is necessary.

Finally, it is noted that the cooling phase is not considered in the current simplified TDA framework. It has been recently recognised that joint failure (mainly tensile rupture) can occur during the cooling stage due to the effect of beam contraction, and nearly all the failure cases happen due to the rupture of fire-affected joints rather than ambient joints. According to the failure criteria defined for this study, collapse is only governed by failure of the surrounding ambient joints, while failure of the fire-affected joint does not necessarily trigger progressive collapse. Considering in effect the similarity of joint rupture at cooling phase and joint shear failure (punching shear) during the heating phase, potential structural failure during the cooling phase can be alternatively simulated via a punching shear analogy. Therefore, the additional consideration of a cooling phase in the simplified TDA may not be necessary, although
it can be simulated easily by monotonically decreasing the temperature beyond the peak value. However, if a structure with smaller column spacing or a larger fire affected area is considered, the surrounding joints can also fail due to elevated temperature. Under such circumstances, performing a cooling phase analysis may be required, since failure can occur in the surrounding joints experiencing a cooling stage, thus triggering progressive collapse. Such scenarios, however, are beyond the scope of this study.
CHAPTER 7

Practical Design Recommendations

7.1 Introduction

While the simplified TDA offers a reliable performance-based analysis method for assessing structural robustness considering localised fire, a comprehensive structural analysis has to be performed, which still requires significant computational and modelling effort. In this respect, alternative prescriptive methods may be more beneficial for design application, at least for the preliminary design stage. For example, instead of analysing the ductility demand/capacity of the floor system for a specific structure under fire, general practical design recommendations on selection of joint detailing, strengthening of structural components, or application of fire protection could be helpful in ensuring a reasonable structural design before a more detailed structural fire analysis is performed.

The main aim of this chapter is to investigate different structural design schemes via parametric studies, thus gaining a more comprehensive understanding of their influence on structural robustness, and enabling the proposal of practical robustness design recommendation for typical steel-composite structural buildings subject to localised fire. This chapter is a natural continuation of the previous performance-based studies, but focuses more on the development of prescriptive recommendations meeting the industrial need for design efficiency and retrofitting guidance. To address this, the eight-storey composite car park considered in the previous chapter is also used as the reference building for this study, where the basic gravity load of 5kN/m² is employed. In the parametric study, the car park is assumed to be designed according to different
schemes and is subjected to the idealised monotonically increased temperature field in accordance with the simplified TDA framework. Two localised fire locations, namely, fire at the ground floor internal column, and fire at the top floor internal column, are considered, and Level B full slab models with shell elements simulating the floor slab are employed. The factors considered in this study as potentially influencing structural robustness against localised fire, and which are therefore varied parametrically, are:

- Influence of fire protection in steel beam
- Influence of fire protection in steel joint
- Influence of fire protection in fire affected column
- Influence of fire protection in slab metal deck
- Influence of column stiffener
- Influence of joint rebar
- Influence of slab reinforcement mesh
- Influence of slab metal deck thickness

### 7.2 Parametric studies

#### 7.2.1 Influence of fire protection in steel beam

Anti-fire coatings are generally employed in structural members in conventional fire design procedures in order to fulfil the codified fire resistance requirements for each individual member. When a single-span steel beam directly exposed to fire is considered, fire protection can significantly decrease its internal temperature, thus efficiently enhancing its strength and reducing its deflection during fire. However, from the perspective of robustness where a significant double-span floor deflection subsequent to column buckling due to localised fire is allowed, the efficiency of fire protection on steel beams has not been previously considered. In order to evaluate the influence of steel beam fire protection on overall structural robustness, two idealised fire protection schemes are proposed, apart from the reference case where no fire protection is applied, namely, half beam protection and full beam protection. For the case of half beam protection, the temperature in steel beams is always halved compared with that in the reference case, while for the full beam protection case, the steel beams are ideally considered under ambient conditions throughout the entire heating procedure.
Figure 7.1 presents the development of the overall shear force transferred by the fire affected steel joint at the ground floor and predicts the punching shear temperature accordingly. The influences of the different steel beam protection schemes on the behaviour of the considered structure for the ground floor and the top floor are illustrated in Figures 7.2 and 7.3 respectively, where the deflection of the fire affected floor and the axial force of the fire affected column are given. Contrary to the conventional recognition expecting the deflections of single-span beams with fire protection to be reduced, the results indicate that applying fire protection on steel beams can lead to larger floor deflections in a double-span manner subsequent to column buckling.

![Fig. 7.1 Influence of beam fire protection on shear response of fire affected connections](image)

This phenomenon implies that increasing local resistance of the heated steel beams in the vicinity of the bucked column via fire protection has no beneficial contribution, or even negative influence on the robustness behaviour of the fire affected floor. This may be explained by the following two reasons. Firstly, the unprotected steel beams exposed to fire can induce considerable axial compressive forces due to thermal expansion, which can be beneficial for enhancing the bending rigidity and the rotational resistance of the joints through ‘pre-compression’, provided that no premature compressive joint failure is initiated. The benefit of thermal expansion can be clearly verified in Figure 7.4, which shows that excluding the steel beam thermal expansion in the structural model can increase the final floor deflection (for the case of fire at the top floor). Secondly,
relatively larger single-span deflections of the fire unprotected beams before column buckling can induce more significant compressive arching effect upon column buckling compared with the fire protected beams, as illustrated in Figure 7.5. This arching effect during column buckling can further increase the compressive force in the steel beams and the joints and as a result may potentially mitigate dynamic effects during column buckling, thus reducing the final floor deflection.

**Fig. 7.2** Influence of beam fire protection on overall structural response for fire at ground floor

**Fig. 7.3** Influence of beam fire protection on overall structural response for fire at top floor
Different fire protection schemes on the steel beams can also greatly affect other aspects of the structural response, such as the column buckling temperatures and the compressive forces transferred to the fire affected column. The influence of the fire protection on column buckling temperature is closely associated with the generated column axial compressive force, as previously indicated in Figures 7.2 and 7.3, which show that the column compressive force is reduced by applying fire protection on the beam. This phenomenon can be also greatly attributed to thermal expansion, as verified in Figure 7.6, where it is shown that excluding the thermal expansion of the steel beam significantly relieves the column compressive force during the heating procedure. This can be caused by uneven thermal expansions between the slab and the beam (thermally expanded slab connected to thermally unexpanded beam) which lead to an upward thermal bowing effect.
Fig. 7.6 Influence of steel beam thermal expansion on column axial compression

Fig. 7.7 Influence of beam fire protection on rebar elongation of surrounding ambient major-axis joint

Since the rupture of the rebars governs the robustness limit state in the considered car park, Figure 7.7 gives the rebar elongations in the surrounding ambient major axis joints. It can be observed that applying fire protection on steel beams tends to increase the final rebar elongations of the surrounding ambient major axis joints, thus indicating a greater potential for progressive collapse. From a perspective of robustness assessment, capacity/demand ratios (CDR) are used in this study to indicate the proximity of the structure to collapse limit states and to quantitatively compare the influence of the different fire protection schemes. The CDR is expressed in terms of the structural
resistance at the point of failure compared to the applied loading, where a ratio exceeding a value of 1.0 indicates a safe structure. The CDR for a system depends on temperature, so a maximum temperature of 1000°C is considered in this study for CDR assessment. Table 7.1 gives the CDRs of the ground floor and the top floor subject to fire with the different fire protection schemes on the steel beams. For fire at the ground floor, it is found that applying fire protection slightly decreases the CDR of the system, particularly under half protection, but the influence is quite small due to more significant influence from the upper ambient floors. On the other hand, fire protection on steel beams has more evident adverse influence on structural robustness for fire at the top floor, where the corresponding CDR is greatly reduced. Therefore, it is concluded that applying fire protection only on steel beams brings no benefit for overall structural robustness and thus is not recommended for typical steel/composite car parks.

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Fire protection on steel beams</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>Full protection</td>
<td>6.58</td>
<td>5.0</td>
<td>1.316</td>
</tr>
<tr>
<td></td>
<td>Half protection</td>
<td>6.45</td>
<td>5.0</td>
<td>1.290</td>
</tr>
<tr>
<td></td>
<td>No protection (Ref.)</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td>Top floor</td>
<td>Full protection</td>
<td>4.78</td>
<td>5.0</td>
<td>0.956</td>
</tr>
<tr>
<td></td>
<td>Half protection</td>
<td>4.80</td>
<td>5.0</td>
<td>0.960</td>
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<tr>
<td></td>
<td>No protection (Ref.)</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
</tbody>
</table>

### 7.2.2 Influence of fire protection in steel joint

Localised fire normally leads to high temperatures in fire affected columns as well as the joints immediately above. According to the simplified TDA analysis discussed in the previous chapter, first joint failure can be found in the fire affected joint under sagging moment immediately after column buckling. In addition, shear failure of the joints under high temperature can lead to punching shear of the fire affected floor, which also potentially increases the potential of successive progressive collapse. Although in the current study progressive collapse is deemed to be triggered only when failure of the surrounding ambient joints occurs, protection on the fire affected joints can effectively avoid punching shear and may also play a positive role in relieving the surrounding ambient joints, thus decreasing the potential for progressive collapse. To
investigate this issue, two joint protection schemes are proposed and compared with the reference case where no fire protection is applied, namely, half joint protection and full joint protection. For the half joint protection case, the temperature in the steel connections in always halved compared with that in the reference case, while for the full joint protection case, the steel connections exposed to fire are ideally considered at ambient temperature throughout the entire heating procedure.

Figures 7.8 and 7.9 show the influence of the different joint fire protection schemes on the deflection of the fire affected floor and the fire affected column axial force for fire at the ground floor and the top floor respectively. For fire at the ground floor, the fire protection on joints has negligible effect on floor deflection immediately after column buckling, which is clearly due to the seven upper ambient floors that dominate the overall structural response. After column buckling, however, fire protection on joints can effectively prevent further increase of the deflection of the fire affected floor associated with punching shear. Regarding fire at the top floor, the fire protection on joints reduces the deflection of the fire affected floor by around 100mm, but it has no influence on column buckling temperature and on the column compressive force during heating. This implies that the column axial force during the heating is not directly linked to the behaviour of the fire affected joints, but more relies on the response of the slab and the steel beams.

![Fig. 7.8 Influence of joint fire protection on overall structural response for fire at ground floor](image-url)
Fig. 7.9 Influence of joint fire protection on overall structural response for fire at top floor

Since the fire affected joints are normally subjected to a considerable sagging bending moment after column buckling, particularly for the top floor, first joint failure can be found in these joints due to first rupture of their lowest bolt-rows, as illustrated in Figure 7.10. Figure 7.11 provides the elongations of the lowest bolt-row of the fire affected major-axis joint under sagging moment with different joint protection schemes, where it can be observed that the elongation of the lowest bolt-rows is significantly reduced due to joint protection. While a considerable reduction of bolt-row elongation is observed, it is noted that the difference between the half and full protection cases is not significant, which implies that applying excessive fire protection on joints may be neither economical nor necessary. Furthermore, the elongation of the lowest bolt-row of the fire affected joint tends to decrease with temperature after column buckling, particularly for fire at the ground floor. This confirms the fact that thermal expansion of
the steel beams can induce pre-compression onto the joints, thus tending to close the gap between the end-plate and the column flange.

Figure 7.11 shows the influence of joint fire protection on elongation of the lowest bolt-row of major-axis joints. It is observed that although applying fire protection on joints can effectively increase their moment resistance under fire and reduce the potential of punching shear, the overall ductility demand of the structure prior to punching shear is not affected. This implies that the surrounding ambient part of the fire-affected floor system plays a more significant role in mitigating progressive collapse compared with the capacity of the heated steel joints immediately above the buckled column, considering the fact that first
local failure of the fire affected joints does not necessarily mean an immediate overall collapse of the structure. Therefore, provided that the temperature is not sufficiently high to induce punching shear, applying fire protection on steel joints is not necessary when a structure is considered under a localised fire that only affects a limited floor area near the column. Of course, if a severe fire is considered, applying fire protection on fire affected joints at lower floor levels is an effective way of preventing punching shear, thus mitigating the risk of progressive collapse.

Table 7.2 provides the CDR’s of the cases of fire at the ground floor and fire at the top floor subject to the different fire protection schemes on joints. As expected, the CDR for the ground floor under fire is increased when joint fire protection is applied, which is mainly due to the avoidance of punching shear. But for the top floor where punching shear is less likely, the improvement is rather limited. It should be noted that an important factor causing negligible benefits from joint fire protection is that fire is localised to a limited area, and most of the surrounding parts of the fire affected floor system as well as adjacent columns are at ambient temperature. For other types of structure, however, the surrounding joints can be subjected to elevated temperature if the column spacing is small enough to allow the localised fire to affect more than two bays. In this case, benefits from fire protection in the surrounding joints would be expected and should therefore be re-evaluated.

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Fire protection on joints</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>Full protection</td>
<td>7.00</td>
<td>5.0</td>
<td>1.400</td>
</tr>
<tr>
<td></td>
<td>Half protection</td>
<td>7.00</td>
<td>5.0</td>
<td>1.400</td>
</tr>
<tr>
<td></td>
<td>No protection (Ref.)</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td>Top floor</td>
<td>Full protection</td>
<td>5.70</td>
<td>5.0</td>
<td>1.140</td>
</tr>
<tr>
<td></td>
<td>Half protection</td>
<td>5.68</td>
<td>5.0</td>
<td>1.136</td>
</tr>
<tr>
<td></td>
<td>No protection (Ref.)</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
</tbody>
</table>

7.2.3 Influence of fire protection in steel column

Since ‘single-span type’ joint failure is not likely to occur before column buckling in full slab models, and provided that first failure of the fire affected joint in shear
(punching shear) is prevented before column buckling, column buckling could be the only and most direct trigger of progressive collapse under localised fire. Conventionally, in order to increase the strength of steel columns under fire, fire protection is applied by means of concrete warp or anti-fire coating materials. This section investigates the influence of column fire protection on structural robustness, where an ideal column fire protection scheme, which halves the column temperature throughout the heating procedure, is assumed to be applied onto the fire affected column to prevent buckling up to a peak floor temperature of 1000°C.

Fig. 7.13 Influence of column fire protection on shear response of fire affected connections

It should be noted that although column buckling can be prevented with fire protection, the risk of progressive collapse still exists due to potential punching shear, as shown in Figure 7.13 which presents the development of the overall shear force transferred by the fire affected joints. It can be observed that punching shear for fire at the top or the ground floor with protected columns occurs at 750°C, which is lower than the reference value (780°C) for fire at the ground floor with an unprotected column. Evidently, this is due to higher overall shear forces transferred by the fire affected joints connected to the unbuckled column compared with the reduced shear force transferred by the joints connected to the buckled column with 300mm floor deflection.
Fig. 7.14 Influence of column fire protection on structural response for fire at ground floor

Fig. 7.15 Influence of column fire protection on structural response for fire at top floor

Figures 7.14 and 7.15 show the influence of column fire protection on the overall structural response (floor deflection and column axial compression) for fire at the ground and top floors, respectively. For fire at the ground floor, although column buckling is prevented due to fire protection, punching shear still occurs and consequently leads to a significant floor deflection which is slightly larger than that observed in the reference case. The larger deflection is due to the fact that the abrupt punching shear in the column protected case starts approximately at a zero deflection before column buckling, which potentially causes more significant dynamic effects than the reference case, where shear punching starts to occur at a deflection of around 300mm after column buckling. As shown by the variation of the column compressive
force for fire at the ground floor, the compressive force that the fire affected column resists is reduced after punching shear (leading to detachment of the fire affected floor); afterwards, the fire affected column only needs to resist the remaining gravity load from the seven upper ambient floor levels.

On the other hand, the behaviour of the floor system at the top floor is similar to that observed for fire at the ground floor with column protection, where first joint failure is governed by punching shear. In addition, the variation of the top floor column axial force is not significantly affected by column fire protection, although more significant thermal expansion is expected in the unprotected column. This indicates that the variation of column axial force during the heating procedure is not mainly related to thermal expansion of the column, but can depend more on the behaviour of the fire affected floor system above the column.

Considering the ductility demand of the surrounding ambient joints, Figure 7.16 presents the influence of column fire protection on the rebar elongation of the surrounding ambient major axis joints. This again confirms that applying fire protection to prevent column buckling does not guarantee structural safety and does not improve structural robustness in the presence of punching shear. Furthermore, the difference between the final rebar elongations at the ground floor and the top floor at the maximum temperature of 1000°C is relatively small. This is because the final resistance mechanisms of the car park at the peak temperature of 1000°C for all the cases subsequent to either column buckling (at the top floor) or punching shear are similar, where the individual fire affected floor system has to resist the gravity load on its own in double span. Table 7.3 gives the CDR’s of the car park subject to localised fire up to a peak floor temperature of 1000°C with different column protection schemes. As observed, the influence of column fire protection on overall structural robustness under such a high temperature, which causes punching shear, is not significant. Therefore, applying fire protection on steel columns only (without applying joint protection) is not recommended, particularly when a server fire is considered.
7.2.4 Influence of fire protection in slab steel deck

It has been well recognised from the previous study that membrane action of the slab is essential in providing significant gravity load resistance after column buckling and thus improving structural robustness. Under localised fire conditions, part of the fire affected floor is significantly heated, which leads to significant strength and stiffness degradation of the local fire-affected area, especially for the steel deck that is directly exposed to fire. An effective way to prevent high temperatures in the slab is to apply anti-fire coating to the steel deck. In order to investigate the influence of slab fire protection on overall structural robustness, two other idealised slab fire protection schemes are considered, namely, half slab protection and full slab protection. For the case of half slab protection, all the temperatures in steel deck and concrete are proportionally halved compared to the reference case, while for the case of full slab
CHAPTER 7 Practical Design Recommendations

protection, the slab including the steel deck is ideally assumed to remain under room temperature throughout the entire heating procedure.

Fig. 7.17 Influence of slab fire protection on shear response of fire affected connections

Figure 7.17 presents the development of the overall shear force transferred by the fire affected steel joint for fire at the ground floor. It can be observed that punching shear is delayed due to the applied fire protection on the slab, particularly for the full protection scheme. This may be because the transverse response of the fire affected floor system with slab fire protection is relatively stiffer than without protection, so under a similar double-span deflection of around 300mm with additional resistance provided by the upper ambient floors after column buckling, the fire affected steel joint in stiffer floors (applied with slab fire protection) is required to resist less shear force than in the unprotected fire floor. Figures 7.18 and 7.19 show the influences of slab fire protection on the overall structural responses for the cases of fire at the ground and top floors, respectively. As anticipated, the slab fire protection reduces the final deflection of the fire affected floor, which is evidently due to the stiffer response exhibited by the protected slab in the absence of material degradation. It can also be seen that the generated column compressive force is not significantly affected by slab protection.
Although slab fire protection can decrease the floor deflection, the rebar elongations of the surrounding ambient major axis joints, which govern the collapse mode of the current structure, are not affected, as shown in Figure 7.20. This may be explained by the fact that significant thermal strain can be induced in the unprotected floor slab, and this tends to close the cracks along the edge of the concrete slab and thus reduce the rebar elongation. In other words, applying slab protection can on one hand relieve the rebar elongation demand through decreasing the floor deflection, but on the other hand tends to increase the rebar elongation near the crack area due to the lack of thermal expansion within the slab.
**Table 7.4 Influence of slab protection on robustness - 1000°C peak temperature**

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Fire protection on slab</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>Full protection</td>
<td>6.58</td>
<td>5.0</td>
<td>1.316</td>
</tr>
<tr>
<td></td>
<td>Half protection</td>
<td>6.37</td>
<td>5.0</td>
<td>1.274</td>
</tr>
<tr>
<td></td>
<td>No protection (Ref.)</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td>Top floor</td>
<td>Full protection</td>
<td>5.65</td>
<td>5.0</td>
<td>1.130</td>
</tr>
<tr>
<td></td>
<td>Half protection</td>
<td>5.28</td>
<td>5.0</td>
<td>1.056</td>
</tr>
<tr>
<td></td>
<td>No protection (Ref.)</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
</tbody>
</table>

Table 7.4 provides the influence of slab fire protection on the CDR’s considering the maximum temperature of 1000°C. As a general remark, no evident improvement of structural robustness is found when the fire protection is applied, and the structure with ‘half slab protection’ can even be more prone to progressive collapse than without slab fire protection.

### 7.2.5 Influence of column stiffener

In the reference car park where column web stiffeners are employed, the overall response of each individual ambient floor (e.g. stiffness under vertical load and the corresponding ductility supply) is almost identical although varied column sizes are adopted at every two floors. This is attributed to identical floor systems that are
employed for all floor levels where the same failure mode is observed (i.e. rupture of rebars at major axis joints under hogging moment), thus the influence of different supporting column sizes on the overall floor response is negligible. In the absence of the column stiffener, however, failure can be governed by the column web in compression before the rupture of rebars in some smaller size columns, as illustrated in Figure 7.21.

This type of failure can cause a complete yielding of the column web in the transverse direction and thus significantly reduce the resistance and ductility of the joints. Furthermore, failure of the column web in compression inevitably reduces the vertical resistance of these ambient perimeter columns and as a consequence increases the potential for a more catastrophic failure of an entire floor due to a successive failure of the supporting columns. This does not conform to the ‘Robustness Limit State’ as for the current study, which assumes no failure of the surrounding columns that are not directly affected by fire. Therefore, a system is deemed to fail when the yield resistance of the column web of any surrounding joint is exceeded. In this respect, the ductility of individual floors supported by different columns without web stiffeners varies; hence the ductility of floor levels with different columns (i.e. HEB220, HEB300, HEB400, and HEB550) need to be considered separately.

It is recalled that for a nonlinear spring representing the above ambient floor system, the response (e.g. stiffness and maximum ductility supply) are determined using the following procedure. Firstly, the gravity loading and an upward point load (exerted at the position of internal column) is initially applied, where the upward point load is used
to represent the supporting internal column, such that the initial floor deflection at the column location is zero. Subsequently, an increasing downward point load is exerted at the same position until one of the surrounding joints exceeds its ductility supply. The load-deflection relationship of each ambient floor can be recorded and incorporated into the spring connected to the column top of the fire affected floor. If more than one ambient floor is considered, the spring response should be obtained through the superposition of the responses of all the upper ambient floors. Employing this approach, the load-deflection relationship for each ambient floor without column stiffeners can be obtained, as shown in Figure 7.22. The corresponding failure load/deflection as well the failure mode of each considered ambient floor are given in Table 7.5.

![Graph showing load-deflection relationship](image)

**Fig. 7.22** Response of an individual ambient floor supported by different unstiffened columns

It is observed that employing different sizes of supporting columns with no web stiffeners has a negligible effect on the overall response of the ambient floors, but has a drastic influence on their ductility supply due to different capacities of the column webs in compression. It is shown that premature failure of the column web in compression governs the collapse mode for floor systems supported by the relatively smaller columns (i.e. HEB 220 and HEB 300 columns) in advance of rebar rupture in the considered building, thus greatly reducing the overall ductility supply of the floor. On the other hand, for larger columns with higher resistance of column web in compression, the ultimate elongation of the rebars in the hogging major-axis joints is exceeded before the failure of unstiffened column web in compression. Accordingly, in the absence of
column web stiffeners, higher floor levels have much less resistance/ductility supply than the lower floors where larger column sizes are employed.

**Table 7.5** Failure response of individual ambient floor systems supported by different unstiffened columns

<table>
<thead>
<tr>
<th>Ambient floor levels</th>
<th>Vertical point load at failure</th>
<th>Maximum vertical deflection</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor levels 1 and 2 (HEB550)</td>
<td>1528kN</td>
<td>570.5mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>Floor levels 3 and 4 (HEB400)</td>
<td>1520kN</td>
<td>566.5mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>Floor levels 5 and 6 (HEB300)</td>
<td>628kN</td>
<td>152.5mm</td>
<td>Failure of column web in compression in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>Floor levels 7 and 8 (HEB220)</td>
<td>452kN</td>
<td>84.9mm</td>
<td>Failure of column web in compression in major axis beam-to-column joint under hogging moment</td>
</tr>
</tbody>
</table>

Obtaining the response of the ambient floors and incorporating them into a nonlinear spring, a system model comprising both the fire affected and the ambient floors (except the top floor where only fire affected floor is considered) can be established. It should be noted that the ductility supply (maximum deformation) offered by the spring representing the upper ambient floors is determined by the least ductile floor. Therefore, for fire at the ground floor, the maximum deflection allowed by the seven upper ambient floors is limited by floor levels 7 and 8 where the smallest column size is employed. Figures 7.23 and 7.24 show the influence of column web stiffeners on the overall structural response under fire for floor levels 1 and 8 respectively. Evidently, premature failure of the column web in compression significantly reduces the robustness of the structure for fire at either the top or the ground floors. For fire at the ground floor, failure of the column web in compression is first found at the surrounding major axis joints under hogging moment at floor levels 7 and 8 along with column buckling. For fire at the top floor, failure of the column web in compression occurs at the major axis joints under hogging moment in a single-span manner even before column buckling. Clearly, this is due to a considerable compressive force (induced by the hogging moment as well as thermal expansion) which leads to the yielding of the unstiffened column web.
Table 7.6 provides the CDR’s for the structure subject to fire at floor levels 1 and 8, where it is clear that the CDR’s in both cases drop far below 1.000, which confirms that the robustness of the considered structure is drastically reduced due to premature failure of the column web in transverse compression in the absence of column web stiffeners. This finding clearly suggests that for the upper floors where smaller column sizes are employed, applying column web stiffeners is essential to ensure structural safety, even if they are deemed as unnecessary under normal loading.
Table 7.6 Influence of column web stiffeners on robustness - 1000°C peak temperature

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Column web stiffener</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>Without stiffener</td>
<td>3.08</td>
<td>5.0</td>
<td>0.616</td>
</tr>
<tr>
<td></td>
<td>With stiffener (Ref.)</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td>Top floor</td>
<td>Without stiffener</td>
<td>1.42</td>
<td>5.0</td>
<td>0.284</td>
</tr>
<tr>
<td></td>
<td>With stiffener (Ref.)</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
</tbody>
</table>

7.2.6 Influence of joint rebars

For the reference building, the rupture of the rebars in the vicinity of the surrounding major axis beam-to-column joints under hogging moments governs the ductility supply of the ambient and fire-affected floors. Therefore, it is expected that strengthening the major axis beam-to-column joints via increasing the allowed rupture elongation of the corresponding rebars can effectively improve the structural robustness against localised fire. In this context, a parametric study is undertaken with three rebar sizes employed for the major axis beam-to-column composite joints, which are 10×8mm rebars, 10×12mm rebars (reference case), and 10×15mm rebars. The properties of the minor axis beam-to-column and the beam-to-beam joints remain unchanged.

According to Anderson et al. (2000), the rupture elongations of the considered 8mm, 12mm, and 15mm diameter rebars are 7.80mm, 11.95mm, and 14.30mm, respectively.
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The corresponding responses of the individual ambient floor are shown in Figure 7.25. It can be seen that the nonlinear response of the ambient floor system is not significantly affected by the joint rebars, this is due to the fact that the increase of rebar diameter in the current model is simulated via locally increasing the reinforcement ratio in the shell elements near the four major axis joints (two hogging and two sagging joints), thus their influence on the overall nonlinear response of the floor system is quite limited. On the other hand, the ductility of the floor is considerably enhanced with the larger rebar diameters due to greater rebar rupture elongation. The maximum load and the deformation capacity that can be accommodated by the ambient floor with the different rebar diameters are given in Table 7.7.

**Table 7.7 Influence of rebar diameter on ambient floor system**

<table>
<thead>
<tr>
<th>Rebar Diameter</th>
<th>Vertical point load at failure</th>
<th>Maximum vertical deflection</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>10×Φ15mm rebar</td>
<td>1840kN</td>
<td>702.6mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>10×Φ12mm rebar (Ref.)</td>
<td>1600kN</td>
<td>597.2mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>10×Φ8mm rebar</td>
<td>1112kN</td>
<td>400.1mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
</tbody>
</table>

**Table 7.8 Influence of rebar diameter on robustness - 1000°C peak temperature**

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Rebar diameter</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>10×Φ15mm rebar</td>
<td>7.21</td>
<td>5.0</td>
<td>1.442</td>
</tr>
<tr>
<td></td>
<td>10×Φ12mm rebar (Ref.)</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td></td>
<td>10×Φ8mm rebar</td>
<td>4.92</td>
<td>5.0</td>
<td>0.984</td>
</tr>
<tr>
<td>Top floor</td>
<td>10×Φ15mm rebar</td>
<td>6.45</td>
<td>5.0</td>
<td>1.290</td>
</tr>
<tr>
<td></td>
<td>10×Φ12mm rebar (Ref.)</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
<tr>
<td></td>
<td>10×Φ8mm rebar</td>
<td>4.27</td>
<td>5.0</td>
<td>0.854</td>
</tr>
</tbody>
</table>

Employing different rebars for the major axis joints changes the ductility supply of the considered structure with negligible influence on its ductility demand. As a result, the capacity-demand ratio is clearly affected, as shown in Table 7.8, which confirms that increasing the rebar diameters can effectively increase the CDR, thus mitigating the...
potential for progressive collapse. It is noted that when the diameter of the rebar
decreases to 8mm, the corresponding CDR’s for fire at both the ground and top floors
drop below 1.000, indicating an increased potential for progressive collapse. In view of
this, it is suggested in design application that joint rebars with sufficient reinforcement
ratios should be utilised to mobilise favourable composite action, even though there
may well be residual system capacity after rebar rupture if the bare steel joint has
additional ductility.

7.2.7 Influence of slab mesh reinforcement

While it was shown in Section 7.2.4 that applying slab fire protection brings no benefit
in mitigating the risk of progressive collapse for the reference car park under localised
fire, an alternative way to improve the slab resistance is to directly strengthen the
ambient resistance of the slab. An evident advantage of such a measure is that the load
resistance is improved not only for the fire-affected floor but also for the upper ambient
floors.

This section studies the influence of mesh reinforcement in the composite floor slab on
the overall structural robustness, where three reinforcement levels are considered,
specifically, 100mm²/m, 250mm²/m (reference case), and 500mm²/m. According to the
method of Anderson et al. (2000), the influence of the local mesh reinforcement on the
rupture rebar elongation of a composite joint is negligible, hence it is assumed that for
all the considered floor systems, the maximum reinforcement elongations are identical
with a value of 11.95mm. Figure 7.26 shows the load-deflection response for an
individual ambient floor system with the three different mesh reinforcement levels,
while Table 7.9 provides the maximum vertical point load resistance and corresponding
vertical deflection for each reinforcement level. An improvement of both stiffness and
ductility supply is observed for the ambient floors with a higher level of mesh
reinforcement. The improvement of stiffness becomes evident at a relatively large
deflection exceeding 200mm. This is due to the fact that under a small deflection, the
resistance mechanism of the floor system is mainly in flexural bending, where the
resistance of 1D composite beams dominates the floor response, where the concrete
flange is mainly under compression. As the deflection becomes larger, increased mesh
reinforcement becomes advantageous in developing membrane action of the slab where
tensile membrane stresses becomes dominant throughout the slab.
Fig. 7.26 Response of an individual ambient floor with various slab meshes

Table 7.9 Influence of slab mesh on ambient floor system

<table>
<thead>
<tr>
<th>Slab mesh</th>
<th>Vertical point load at failure</th>
<th>Maximum vertical deflection</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>100mm²/m mesh</td>
<td>1840kN</td>
<td>646.6mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>250mm²/m mesh (Ref.)</td>
<td>1600kN</td>
<td>597.2mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>500mm²/m mesh</td>
<td>1432kN</td>
<td>555.4mm</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
</tbody>
</table>

Obtaining the response of the ambient floors, the overall structure is investigated employing the simplified TDA framework. Figure 7.27 presents the development of the overall shear force transferred by the fire affected steel joint at the ground floor, and Figures 7.28 and 7.29 show the floor deflection and column axial force of the structural model subject to fire at the ground and top floors, respectively. It can be seen that different slab meshes have negligible influence on the shear force resisted by the fire affected connections, thus the same temperature is achieved for the three cases to trigger punching shear. For fire at either the ground or the top floor, the final deflection of the fire affected floor is considerably affected by the slab meshes, where decreased floor deflections are observed in the structure with greater slab reinforcement due to increased load resistance of all floor levels at large displacement. On the other hand, the column axial force is clearly unaffected by the slab meshes, since the development of column buckling under localised fire occurs at relatively small displacements.
Fig. 7.27 Influence of slab mesh on shear response of fire affected connections

Fig. 7.28 Influence of slab mesh on structural response for fire at ground floor

Fig. 7.29 Influence of slab mesh on structural response for fire at top floor
Considering the ductility demand of the joints, Figure 7.30 presents the rebar elongation of the surrounding ambient major axis joints under hogging moments in the fire affected floor system. The obtained curves clearly show that employing a heavier slab mesh reduces the ductility demand on rebars, thus enhancing structural robustness. As observed in Table 7.10, the CDR of the considered structure at the maximum temperature of 1000°C is effectively increased when heavier slab meshes are employed.

*Fig. 7.30 Influence of slab mesh on rebar elongation of ambient major-axis joint*

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Slab mesh</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>100mm²/m mesh</td>
<td>6.12</td>
<td>5.0</td>
<td>1.224</td>
</tr>
<tr>
<td></td>
<td>250mm²/m mesh</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td></td>
<td>500mm²/m mesh</td>
<td>7.10</td>
<td>5.0</td>
<td>1.420</td>
</tr>
<tr>
<td>Top floor</td>
<td>100mm²/m mesh</td>
<td>5.15</td>
<td>5.0</td>
<td>1.030</td>
</tr>
<tr>
<td></td>
<td>250mm²/m mesh</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
<tr>
<td></td>
<td>500mm²/m mesh</td>
<td>6.58</td>
<td>5.0</td>
<td>1.316</td>
</tr>
</tbody>
</table>

7.2.8 Influence of steel deck thickness

Steel decks employed in composite structures not only act as formworks during the design stage of concrete casting, but also offer additional resistance for completed floor systems, particularly along the direction of ribs. This section studies the influence of slab metal deck thickness on the resistance of the floor systems as well as the overall
structural robustness. Three deck thicknesses are considered, which are 0.5mm, 1.0mm (reference case) and 1.5mm, with other geometric properties of the slab remaining unchanged. Employing the three different deck thicknesses, Figure 7.31 provides the load-deflection response of an individual ambient floor, and the maximum load resistance and associated deflection are listed in Table 7.11. It is observed that both the resistance and the ductility supply is improved for the ambient floors with thicker steel deck, but similar to the previous discussion on slab meshes, the improvement only becomes evident when a relatively larger deflection is developed (i.e. over 200mm).

![Response of individual ambient floor with various slab deck thicknesses](image)

**Fig. 7.31 Response of individual ambient floor with various slab deck thicknesses**

<table>
<thead>
<tr>
<th>Steel deck thickness</th>
<th>Vertical point load at failure (kN)</th>
<th>Maximum vertical deflection (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5mm</td>
<td>1328</td>
<td>529.9</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>1.0mm (Ref.)</td>
<td>1600</td>
<td>597.2</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
<tr>
<td>1.5mm</td>
<td>1792</td>
<td>622.5</td>
<td>Rupture of rebar in major axis beam-to-column joint under hogging moment</td>
</tr>
</tbody>
</table>

After incorporating the obtained response of the ambient floor into the nonlinear spring, the fire affected floor system with the spring is then investigated under the simplified TDA temperature distribution. Figure 7.32 presents the development of the overall shear
force transferred by the fire affected steel joints at the ground floor. Figures 7.33 and 7.34 show the floor deflection and column axial force of the structure subject to fire at the ground and top floors, respectively. It is shown that the column axial force is not affected by deck thickness, but the floor deflection after column buckling or after punching shear is greatly affected.

Fig. 7.32 Influence of deck thickness on shear response of fire affected connections

Fig. 7.33 Influence of deck thickness on structural response for fire at ground floor
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Fig. 7.34 Influence of deck thickness on structural response for fire at top floor

Fig. 7.35 Influence of deck thickness on rebar elongation of ambient major-axis joint

With respect to the ductility demand, the rebar elongation of the surrounding ambient major axis joints under hogging moments in the fire affected floor is presented in Figure 7.35. The obtained rebar elongations clearly show that employing a thicker steel deck can effectively reduce rebar elongations. It is found that when the thickness of the steel deck decreases to 0.5mm (half of the reference value of 1.0mm), the rebar elongation of the system for fire at the top floor exceeds the rupture limit, which indicates a high potential for progressive collapse. Table 7.12 provides the CDR’s of the structure with the three different thicknesses of the steel deck. These results are in line with the predictions of the rebar elongations, demonstrating that the steel deck plays an...
important role in enhancing structural robustness through improved resistance for both ambient and fire affected floors.

<table>
<thead>
<tr>
<th>Fire locations</th>
<th>Deck thickness</th>
<th>UDL capacity (kN/m²)</th>
<th>UDL basic load (kN/m²)</th>
<th>Capacity-demand ratio (CDR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground floor</td>
<td>0.5mm deck thickness</td>
<td>5.86</td>
<td>5.0</td>
<td>1.172</td>
</tr>
<tr>
<td></td>
<td>1.0mm deck thickness (Ref.)</td>
<td>6.59</td>
<td>5.0</td>
<td>1.318</td>
</tr>
<tr>
<td></td>
<td>1.5mm deck thickness</td>
<td>7.23</td>
<td>5.0</td>
<td>1.446</td>
</tr>
<tr>
<td>Top floor</td>
<td>0.5mm deck thickness</td>
<td>4.88</td>
<td>5.0</td>
<td>0.976</td>
</tr>
<tr>
<td></td>
<td>1.0mm deck thickness (Ref.)</td>
<td>5.67</td>
<td>5.0</td>
<td>1.134</td>
</tr>
<tr>
<td></td>
<td>1.5mm deck thickness</td>
<td>6.62</td>
<td>5.0</td>
<td>1.324</td>
</tr>
</tbody>
</table>

7.3 Conclusions

This study investigates the influence of different structural design schemes on the robustness of structures subject to localised fire. The main aim is to investigate the prospects of prescriptive guidance and key parameters in the robustness design of typical steel-composite structural buildings subject to localised fire. With this aim, this study has focused on the evaluation of eight design schemes that potentially affect structural robustness, namely, fire protection on steel beam, fire protection on steel joint, fire protection on fire affected column, fire protection on slab metal deck, column stiffener arrangement, joint rebar arrangement, level of slab reinforcement mesh, and thickness of slab metal deck. Simplified TDA analysis allowing a maximum temperature of 1000°C is conducted, where only the heating phase is considered. Two fire locations, namely, fire at the ground floor near an internal column and fire at the top floor near an internal column, are investigated. The following conclusions can be drawn from the results of the cases studied.

- Although the design strategy of applying fire protection has been accepted as an effective way to increase the resistance and limit the deformation of structural members in conventional design applications considering fully-developed fires, this is not the case for the consideration of robustness against localised fire conditions, where the fire affected zone is also kept localised. As found from this study, applying fire protection on steel beams and steel decks has limited
beneficial effect on overall structural robustness, considering the fact that only failure of the surrounding ambient joints can lead to progressive collapse. Under certain circumstances, fire protection may even lead to an undesirable reduction in overall resistance, which can be due to the elimination of thermal expansion.

- Applying fire protection on joints can be effective in avoiding punching shear under an intermediate to severe fire. Due to the fact that punching shear is more likely to occur at lower floors, fire protection on joints may bring no benefit for fire at higher floor levels even under high temperatures (above 750°C). On the other hand, considering a less severe fire with a maximum temperature of less than 500°C (where punching shear is not likely to occur), the influence of joint fire protection can be negligible regardless of the fire affected floor level.

- Although column buckling is normally the most direct trigger for progressive collapse under localised fire, sufficient structural robustness cannot be guaranteed through applying fire protection only to the steel columns due to the risk of shear failure (punching shear) of the unprotected steel connections. However, applying fire protection on joints and columns simultaneously can be highly effective.

- Employing column web stiffeners in relatively smaller columns provides a significant enhancement of progressive collapse resistance through eliminating the potential of undesirable premature failure of the column web in compression. Therefore, for multi-storey or tall buildings where the beam depth is significant and various column sizes are employed at different floor levels for economical purposes, adding column web stiffeners is highly recommended particularly for the columns at higher floor levels.

- The provision of additional reinforcement bars over the hogging moment regions of joints can effectively improve the overall structural robustness for the reference car park where rupture of the rebars generally governs the collapse mode. Of course, upper limits on the amount of rebars need to be determined because excessive reinforcement may also trigger brittle failure mechanisms of local buckling in the compression contact regions of the steel beams, i.e. compressive failure of beam web/flange, especially when no column web stiffeners are applied.
• Strengthening the resistance of slab through employing heavier reinforcement meshes or thicker steel deck thicknesses is adequately effective, particularly at large floor deflections where membrane action is well developed. Recognising the contribution from the upper ambient floors, one obvious advantage of this scheme over applying fire protection is its beneficial influence on all the affected floors including the fire affected floor and the upper ambient floors.

Based on the above findings which form the basis of the practical design recommendation framework, it is further concluded that several schemes / scheme combinations can be effective in improving structural robustness under localised fire, irrespective of the location of the fire, maximum temperature and modelling assumptions. The effective schemes / scheme combinations are: 1) applying column fire protection with joint fire protection, 2) increasing the amount of joint reinforcement with the addition of column web stiffeners, 3) increasing the amount of slab mesh, and 4) increasing the steel deck thickness. As a general remark on the potential applicability of these findings to serve the industry, the obtained effective design schemes / scheme combinations can be readily implemented in design practice during the preliminary design stages. Towards enhanced efficiencies, these design scheme combinations have the potential to constitute an effective prescriptive guidance for robustness design against localised fire. Of course, the final design decisions should also take account of the cost of each available strategy, and should be supplemented with detailed checks using a quantitative framework such as the simplified TDA presented in the previous chapter or the TIA developed in the next chapter. Finally, it is worth pointing out that the practical outcomes are specific to the case of localised fire near an internal column where floor continuity is considered on the various sides. For other cases, e.g. fire near external columns, a similar analysis approach can be employed to obtain corresponding results.
CHAPTER 8

Temperature-Independent Approach (TIA)

8.1 Introduction

While the simplified TDA has been shown to be capable of providing a reliable performance-based robustness assessment procedure for multi-storey buildings subject to localised fire, the definition of the fire affected area and the maximum expected temperature is still required. Accordingly, the simplified TDA can be deemed only as a ‘semi-event-independent’ approach. On the other hand, through the parametric study conducted in Chapter 7, a basic understanding of the influence of different design parameters on structural robustness has been gained, and practical design recommendations have been proposed. These design recommendations are mainly prescriptive and thus not performance-based. Towards a more practical and comprehensive design approach which is event-insensitive and at the same time performance-based, an alternative robustness assessment approach is proposed in this chapter, namely, a Temperature-Independent Approach (TIA).

Under high temperatures, certain parts of steel components in floor systems can lose their strength considerably, thus their contribution to progressive collapse resistance becomes rather limited. It was shown from the previous study that for most scenarios, the axial resistance of the column can be completely lost at elevated temperature. In light of this fact, and inspired by the idea of the event-independent ‘sudden column loss’ scenario currently adopted in some of the guidelines for progressive collapse assessment (GSA, 2003; DoD, 2009), the TIA is developed to offer an event-independent structural
robustness assessment in the sense that the maximum temperature and the temperature distribution are considered to be unknown (as shown in Figure 8.1). Therefore, the TIA model does not require thermal analysis, and can provide a simplified robustness assessment procedure regardless of temperature, thus adopting a similar basis to typical robustness provisions where event-independent local damage scenarios are employed. The proposed TIA is an extension of the application of the multi-level ductility-centred approach (Izzuddin et al., 2008; Vlassis et al., 2008) developed at Imperial College London for assessing the robustness of steel/composite buildings subject to sudden column loss scenarios. An important benefit of the multi-level ductility-centred approach is its convenience for conducting simplified dynamic assessment using the principle of energy balance instead of directly undertaking complex nonlinear dynamic analysis. Similar to this approach, the TIA also assumes a column loss scenario, while additionally considering other factors exclusively associated with localised fire scenarios, e.g., degraded performance of fire affected floor, and residual column resistance during buckling.

![Fig. 8.1 Illustration of TDA and TIA models](image)

In order to better understand the background and the principle of the TIA, the ambient multi-level ductility-centred approach for sudden column loss scenarios is recalled and briefly introduced in Section 8.2, where a single double-span floor system extracted from the reference car park presented for the preceding TDA analysis is employed as an example to illustrate this approach. Afterwards, the TIA is elaborated in Section 8.3 as an extension of the ductility-centred method for localised fire conditions. This is followed by presenting design examples for the illustration of the TIA, where the
previously discussed reference car park is employed for the case study. In order to verify the reliability of the TIA, the behaviour of the reference structure obtained from the TIA is compared with the TDA predictions. Accordingly, both the merits and potential shortcomings of the TIA are identified.

### 8.2 Ductility-centred method for sudden column loss

The ductility-centred method is a performance-based approach that can be easily applied at various levels of structural idealisation (Izzuddin et al., 2008), as illustrated in Figure 8.2. At the highest model level, the affected bay of the multi-storey building with appropriate boundary conditions is considered, as shown in Figure 8.2(a). Provided that the surrounding columns can resist the redistributed load, only the floors above the lost column need to be considered and the model is reduced to that shown in Figure 8.2(b). If the affected floors have identical structural geometry and gravity loading, a further reduced model consisting of an individual floor system may be considered, as illustrated in Figure 8.2(c). Finally, ignoring 2D effects (e.g. membrane action) of the floor slab, the model of Figure 8.2(c) can be reduced to a grillage model with each individual steel/composite beam resisting appropriate proportions of the gravity load, as shown in Figure 8.2(d). Since previous studies indicate that grillage models may underestimate the progressive collapse resistance of a structure due to the neglect of 2D slab effects, this section only introduces the application of the ductility-centred method on models employing 2D slab elements.

Sudden removal of a column is in effect close to suddenly applying the gravity load on the same structure in the absence of the column at the beginning, especially when significant displacements can be sustained by the structure as a result. This sudden application of gravity loading is associated with dynamic effects, where the maximum dynamic response / the ductility demands must be accommodated to avoid progressive collapse. In this context, the ductility-centred approach is comprised of three basic components, namely, *nonlinear static response*, *simplified dynamic assessment*, and *ductility assessment*. Employing this framework, it is only necessary to determine the nonlinear static load-displacement response of the double-span structure/substructure in the absence of the damaged column. Subsequently, the maximum dynamic displacement after sudden column loss is determined using a simplified dynamic assessment approach, which utilises the principle of energy balance to transform the
nonlinear static response to a nonlinear dynamic response. The last step establishes the maximum ductility demand (e.g. maximum dynamic deflection) and its comparison against the ductility supply. Employing an illustrative example of the single double-span car park floor system, the three basic components of the ductility-centred approach are presented as follows.

![Sub-structural model levels for progressive collapse assessment](image)

**Fig. 8.2** Sub-structural model levels for progressive collapse assessment (Izzuddin, et al., 2008)

- **Nonlinear static response**

The nonlinear static response for a substructure subject to sudden column loss is normally expressed by a gravity load vs. deflection relationship, which can provide a measure of the energy absorption characteristics of the floor, as expressed by:

\[
\delta U = \delta W = \alpha P \delta u_s
\]  

(8.1)

where \(\delta U\) is the increment of absorbed strain energy, \(\delta W\) is the increment of the external work, \(P\) is the overall gravity load within the affected bay, \(\delta u_s\) is the increment of the deflection at a reference point, and \(\alpha\) is a non-dimensional weighting factor based
on load distribution on the floor, and may depend on the incremental floor deformation mode. For the 32m×20m double-span individual floor system (i.e. in line with the model level shown in Figure 8.2(c)) employed in the reference car park, as shown in Figure 8.3, a uniformly distributed gravity load can be associated with an incremental plastic deformation mode, hence the value of $\alpha$ may be taken as (Izzuddin et al., 2008):

$$\alpha = 0.25 \quad (8.2)$$

**Fig. 8.3** Single floor system of reference car park

The nonlinear static $\alpha P - u_s$ curve of the considered single ambient floor system subject to internal column loss is illustrated in Figure 8.4. The area below the nonlinear static $\alpha P - u_s$ curve is equal to the strain energy absorbed by the deformed floor system up to a specific deflection of $u_{s,i}$.

**Fig. 8.4** Nonlinear static response of individual ambient floor system
• *Simplified dynamic assessment*

For a given nonlinear static response, the maximum dynamic ductility demand of the floor subject to the equivalent sudden application of gravity load can be obtained from energy balance between the work done by the external load and the internal energy dissipated in the deformed floor. Initially at small deflections, the suddenly applied gravity load exceeds the static structural resistance, and the differential work done over the incremental deformations is transformed into additional kinetic energy, thus leading to increasing velocities. As the deformation increases further, the static resistance would eventually exceed the gravity load, and the differential energy absorbed leads to a reduction in the kinetic energy, thus leading to decreasing velocities. With the assumption that the deflected floor system is dominated by a single deformation mode, which is likely to be the case for a sudden column loss scenario, the maximum dynamic deflection $u_{d,i}$ is achieved for a gravity load level $P_i = \lambda_i P_o$ when the kinetic energy is reduced back to zero. At this point of equilibrium, the external work done $W_i$ by the suddenly applied gravity load $P_i$ is identical to the internal strain energy dissipated by the structure, as illustrated in Figure 8.5 (a) and (b) for two different load levels, which is expressed by:

$$W_i = \alpha_i P_i u_{d,i} = U_i = \int_0^{u_{d,i}} \alpha_i P du_s$$

(8.3)

This leads to a pseudo-static response, which depicts the maximum dynamic displacement corresponding to a specific level of suddenly applied gravity load, as shown in Figure 5(c).

![Fig. 8.5 Simplified dynamic assessment (Izzuddin et al., 2008)](image_url)

For the considered individual double-span floor system subject to sudden internal column loss, the maximum dynamic deflection can be predicted using the simplified
dynamic assessment approach. The gravity load is taken as 5kN/m\(^2\) (2.5kN/m\(^2\) dead load + 2.5kN/m\(^2\) live load), and can be alternatively expressed as \(\alpha_P = 0.25 \times 5\text{kN/m}^2 \times 32\text{m} \times 20\text{m} = 800\text{kN}\). As shown in Figure 8.6, the static deflection \(u_s\) of the floor is 224mm under the gravity load of 5kN/m\(^2\). Based on the simplified dynamic assessment approach, the equivalence between external work and internal energy is achieved when the two hatched areas become identical. Employing this approach, the maximum dynamic deflection of the considered floor subject to sudden column loss is 468mm, which is approximately twice of the static value. For comparison purposes, the maximum dynamic deflection is also obtained using detailed nonlinear dynamic analysis through ADAPTIC, and the result is 445mm which is very close to the simplified dynamic assessment prediction. The slight difference can be attributed to the assumption of a single deformation mode of the floor system, which ignores other less significant deformation modes.

![Fig. 8.6 Simplified dynamic assessment of floor subject to sudden column loss](image)

- **Ductility assessment**

The aim of the ductility assessment stage is to compare the ductility demand (e.g. maximum dynamic deflection) of a structure with its inherent ductility supply. A structure is deemed to fail the robustness requirements when its ductility demand exceeds the ductility supply. When the surrounding columns have sufficient resistance to sustain the redistributed load, the Robustness Limit State (RLS) adopted in the TDA
models can be also employed here for determining the maximum ductility supply of an ambient structure/substructure subject to sudden column loss. According to the predefined RLS, progressive collapse is attributed to ductility failure of the first surrounding ambient joint on any considered floor, thus failure criteria are defined in terms of whether the ductility limits of the surrounding joints are exceeded. A reliable component method for predicting the joint ductility has already been thoroughly discussed in the previous chapters.

For the current individual floor model example, first failure mode is governed by the rupture of the rebars in the surrounding major axis beam-to-column joints; accordingly, the maximum ductility supply is around 566mm in deflection. This maximum ductility supply is larger than the dynamic ductility demand of 468mm in deflection predicted by the simplified dynamic assessment approach, thus indicating a favourable robustness of the floor system. In order to obtain the maximum dynamic deflections associated with different levels of gravity load, a pseudo-static curve can be constructed, as shown in Figure 8.7. Based on the ductility supply of 566mm in deflection, a maximum gravity load resistance $\alpha P$ of 912kN (5.7kN/m$^2$ UDL) is easily determined, and the corresponding value of the capacity-demand ratio (CDR) is thus obtained as 1.14 for this floor system under a sudden column loss scenario, assuming a gravity load of 5kN/m$^2$.

![Fig. 8.7 Pseudo-static response and ductility supply](image-url)
8.3 Modified ductility-centred method for localised fire – TIA

The ductility-centred method (Izzuddin et al., 2008) outlined in the previous section offers a practical and reliable framework for assessing the robustness of structures subject to sudden column loss scenarios. In this method, it is assumed that the targeted column fails over a very short duration in comparison with the response period of the rest of the structure, which is realistic for columns subject to blast or impact. For such cases, sudden column loss offers an event-insensitive yet realistic scenario of actual structural robustness. For structures subject to localised fire, the fire affected column is eventually lost due to buckling if the temperature is sufficiently high, where it was shown in the previous chapter that column buckling can be associated with significant dynamic effects. This bears some similarity to sudden column loss scenarios, thus providing the possibility of extending the application of the ductility-centred method to fire hazard conditions.

The ambient ductility-centred method for sudden column loss is not directly applicable to structures under localise fire scenarios, so appropriate amendments and improvements are required. In this respect, two issues have to be addressed which are not directly associated with typical sudden column loss scenarios but need to be considered for localised fire situations, namely, 1) degraded performance of fire affected floor, and 2) Residual column resistance during buckling. Incorporating considerations for these two aspects, the TIA, which is a modification of the ductility-centred method, is applicable to the robustness assessment of structures exposed to localised fire.

8.3.1 Degraded performance of fire affected floor

Ambient floors in TIA models should be treated in the same way as in the TDA models, so the main difference between the TDA and the TIA structural models is the treatment of the fire affected floor. For the fire affected floor considered in the TDA model, idealised temperature distributions are applied onto the structural members (including the column, beam, joint, and slab) within a predefined heating zone, and then elevated temperature structural analysis is undertaken over a temperature domain. While this approach can reliably predict the structural degradation of the fire affected floor system and the buckling temperature of the fire affected column, the analysis still depends on foreseen fire information such as the affected heating area and the potential peak
temperature. Towards a more practical approach, robustness assessment should ideally be related to the design of the structures under unforeseen extreme events, such that initial local damage does not propagate to an extent disproportionate to the original cause. In order to accommodate the event-insensitivity requirement into the TIA framework, the structural components / component parts that are directly exposed to fire (and consequently lose their load resistances) are treated as initial local damage and are thus completely removed. The remaining structural components are considered as intact under ambient conditions, as illustrated in Figure 8.8. This assumption forms the basis of the simplification strategy of the fire affected floors in the TIA framework. Details of the member removal strategies considered in this study are explained as follows.

![Diagram](image)

**Fig. 8.8** Illustration of structural member removal for TIA fire affected floor

1) The centre of localised fire is in line with the axis of the considered fire affected column. This assumes that the unforeseen local damage emanates from the location of the fire affected column and propagates symmetrically to adjacent structural members.
2) Considering robustness requirements, the column is assumed to be completely removed in TIA models. This is reasonable because the maximum temperatures achieved in fire accidents are usually unknown, so completely removing the column is in line with the event-independent initial damage strategy.

3) Previous studies on joints indicate that the rotational and axial stiffness as well as the ultimate bending resistance of steel joints can be greatly impaired under high temperatures, so in the current TIA model, the rotational and axial resistance of the steel connections exposed to fire are conservatively considered as ‘removed’ completely. However, the shear resistance of steel connections is assumed to remain adequate, thus the reduced fire affected floor is assumed to be capable of transferring load to the above ambient structure, thus allowing synergistic deflections between the degraded fire floor and the upper ambient floors.

4) For the fire affected floor slab, the concrete remains under ambient conditions, while parts of the steel beam adjacent to the removed column are assumed to lose their resistance completely, so they are also removed. The concept of removing parts of the fire affected beams are in line with the recommendations of Implementation of Eurocodes Handbook 5 (2005) for car parks under localised fires, which suggests that ‘the part of the steel beam above the length of the burning car can be completely neglected, where the remaining slab is supported by the cantilever part of the continuous composite beam’. In this study, three different lengths of steel beams are considered to be removed (0.0H, 0.75H and 1.5H), where H is the height of the column. No slab metal deck is removed in the current TIA model.

In order to illustrate the response of degraded TIA fire affected floors, the fire affected floor system considered in the reference car park is employed. Based on the above assumptions, the degraded TIA fire affected floor model with the three different removed lengths of the steel beams are established in ADAPTIC (Izzuddin, 1991), namely, 0.0H, 0.75H and 1.5H. Figure 8.9 shows the ADAPTIC floor system models with the removed beam lengths of 0.75H and 1.5H. The static load-deflection responses ($\alpha P - u$ relationship) of the TIA floor models are given in Figure 8.10, where $\alpha$ is the weighting factor taken as 0.25 for the floor system, and P is the overall gravity load.
Evidently, the load resistance of the degraded double-span floor systems subject to the removal of the steel beams and connections is much lower than for the non-degraded ambient floor. This suggests that removing the fire-affected structural components can indeed reduce the overall resistance of the floor system in double-span. Moreover, it is found that the responses of the TIA system models with 0.0\(H\), 0.75\(H\), and 1.5\(H\) removed beam lengths are very close, which indicates that the removed beam length has limited influence on the resistance of the degraded floor. This can be explained by the fact that after the removal of the bending and axial resistance of the fire-affected steel connections, the resistance of the degraded floor system is mainly exhibited through 2D membrane action in the slab as well as the resistance of the beams acting in a cantilever manner. Therefore, further removal of steel beam parts in the vicinity of the removed connections does not significantly change the load resisting mechanism of the degraded
CHAPTER 8 Temperature-Independent Approach (TIA)

floor, thus leading to similar responses for the three degraded floor systems (with 0.0H, 0.75H, and 1.5H beam length removal). Therefore, in order to reflect the floor performance without loss of generality, a degraded floor system with only the fire affected steel connections removed (i.e. 0.0H) can be employed to represent the fire affected floor within the TIA framework.

In order to justify the rationale behind the proposed assumptions for individual TIA fire affected floors, the static deflections of the TDA floor system under different levels of gravity load are compared with the static load-deflection relationship of the TIA degraded floor, as shown in Figure 8.11. The peak temperature for the TDA floor is taken as 700°C, which is a typical peak temperature achieved in a localised fire accident with four burning cars around an internal column, as observed in Chapter 5. In order to obtain an actual static deflection of the TDA fire affected floor under gravity loadings, vertical support is applied first at the mid-span to replicate the fire affected column before buckling. Then the idealised TDA temperature distribution is applied onto the floor system. When the maximum temperature is attained (i.e. 700°C in this case), the mid-span support is slowly removed, and the final deflection of the double-span TDA fire affected floor is obtained. As clearly found from Figure 8.11, the TIA degraded floor gives a close prediction of deflection, thus indicating good correlation between the TDA fire affected floor and the TIA degraded floor.

![Fig. 8.11 Static response of TDA and TIA fire affected floor](image)

FIG. 8.11 Static response of TDA and TIA fire affected floor
8.3.2 Residual column resistance during buckling

The ambient ductility-centred approach (Izzuddin et al., 2008) offers a practical and reliable robustness assessment framework for sudden loss of the damaged column. However, extreme events such as blast, impact, and fire do not usually lead to a complete and immediate column loss. Recent work undertaken by Gudmundsson and Izzuddin (2009) suggested that sudden column loss tends to provide an upper bound on the structural deformation demands, which can be approached as the level of extreme loading on the affected column becomes sufficiently large.

Considering a structure under a localised fire, vertical resistance of the fire affected column starts to decrease when the column buckling temperature is achieved. From the discussion in Chapter 6 on the reference car park under localised fire, the arrested floor deflection after complete loss of the fire affected column often falls between two idealised bounds, namely, ‘sudden column loss’ and ‘static column loss’. If the column completely loses its resistance over a very short duration, the structural performance will be close to a sudden column loss response. On the other hand, the structural performance can be close to a full static response when column buckling occurs over a comparable duration to the response of the remaining structure. When the resistance of the fire affected column is not lost immediately, it continues to provide significant resistance during buckling, which leads to a reduced maximum floor deflection compared to the case of sudden column loss.

From the perspective of energy balance, the energy transformation procedure starting from initial column buckling can be described as follows. As the floor system starts to deflect, external work is done by the gravity load while strain energy is absorbed by the deflected floor systems as well as the deflected column. The static resistance of the floor system together with the residual static resistance of the column can be less than the external load, so the differential work done over the incremental deformations is transformed into additional kinetic energy, thus leading to increasing velocity. The energy dissipated by the buckled column may cease at a relatively early stage, depending on the column response characteristics. As the floor deflection increases, the static resistance of the floor system starts to exceed the gravity load, and the differential energy absorbed reduces the kinetic energy, thus leading to decreasing velocity. The maximum dynamic deflection is achieved when the kinetic energy is reduced back to
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zero. At this point of equilibrium, the external work done by the gravity load is identical to the internal strain energy dissipated by the overall structure (including both the floor system and the column), which can be expressed by:

\[ W_i = U_{f,i} + U_{c,i} \]  

(8.4)

where \( W_i \) is the external work done by gravity load \( P_i \), \( U_{f,i} \) is the strain energy dissipated by floor systems, and \( U_{c,i} \) is the additional strain energy stored in buckled column. It is clear that the major difference between Eq. (8.4) and Eq. (8.3) used in the original ductility-centred method for robustness assessment (Izzuddin et al., 2008) is the additional consideration of the energy term \( U_{c,i} \), which potentially mitigates the dynamic effects. Importantly, the consideration of the residual column resistance as a contribution to the energy balance expression extends the ductility-centred approach effectively to account for column damage caused by localised fires.

In order to verify the rationale behind Eq. (8.4), the ambient double-span floor system discussed in Section 8.2 (as shown in Figure 8.3) is employed here to predict the maximum dynamic response, but considering a finite duration for column loss instead of an idealised sudden column loss scenario. The maximum deflections obtained from detailed dynamic analysis are then compared with those predicted by Eq. (8.4). The column loss characteristics are idealised here using a linearly reducing resistance force for the column, as shown in Figure 8.12. Accordingly, the work done by the mid-span support force during the whole removal procedure would be in effect identical to the strain energy \( U_{c,i} \). Various time durations of support removal (leading to different maximum dynamic deflections) are assumed, namely, 0.0 second (sudden loss), 0.2 second, 0.5 second, and 1.0 second. Two levels of gravity UDL are considered, which are 5kN/m² and 7kN/m². Under the two levels of gravity loading the dynamic responses obtained from the detailed dynamic analysis in ADAPTIC are shown in Figures 8.13 and 8.14, respectively.
Alternatively, the simplified dynamic assessment approach considering energy balance is employed to predict the ductility demand of the individual double-span floor system subject to different durations of support removal, as expressed by:
CHAPTER 8 Temperature-Independent Approach (TIA)

\[ W_i = U_{f,i} + W_{\text{support}} \]  

(8.5)

where \( W_i \) is the external work done by the considered gravity load \( P_i \), \( U_{f,i} \) is the strain energy dissipated by the floor system, and \( W_{\text{support}} = U_{c,i} \) is the work done by the mid-span support force during the whole support removal procedure, which can be seen as in effect similar to a column buckling process. In Eq. (8.5), \( W_i \) and \( U_{f,i} \) can be easily obtained through the method discussed previously in Section 8.2, while \( W_{\text{support}} \) is equal to the area enclosed by the effective column resistance curve, as shown in Figure 8.15.

![Support force vs. vertical displacement relationships](image)

**Fig. 8.15** Support force vs. vertical displacement relationships

Employing this simplified method based on Eq. (8.5), the maximum dynamic deflections are obtained and listed in Tables 8.1 and 8.2 for 5kN/m\(^2\) and 7kN/m\(^2\) UDL, respectively. The static deflections and the dynamic deflections obtained from the detailed dynamic analysis using ADAPTIC are also provided for comparison. Favourable correlation is clearly observed with small discrepancies below 8%. This indicates that the proposed energy equivalence method is sufficiently accurate, thus confidence is gained to utilise this method within the TIA framework. Furthermore, it can be observed that the dynamic deflections tend to decrease with the support removal duration, and all the corresponding values fall between the static deflection and the dynamic deflection obtained with sudden support loss. This again confirms the hypothesis that the potential short-term residual column resistance immediately after buckling can lead to a reduction of the dynamic effect, thus leading to a reduced maximum floor deflection.
Table 8.1 Influence of support removing durations on dynamic response – 5kN/m² UDL

<table>
<thead>
<tr>
<th>Load removal duration</th>
<th>Static deflection</th>
<th>Dynamic deflection-Detailed analysis</th>
<th>Dynamic deflection-Simplified method</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0s</td>
<td>224mm</td>
<td>445mm</td>
<td>468mm</td>
<td>5.16%</td>
</tr>
<tr>
<td>0.2s</td>
<td>224mm</td>
<td>404mm</td>
<td>418mm</td>
<td>3.47%</td>
</tr>
<tr>
<td>0.5s</td>
<td>224mm</td>
<td>319mm</td>
<td>331mm</td>
<td>3.76%</td>
</tr>
<tr>
<td>1.0s</td>
<td>224mm</td>
<td>290mm</td>
<td>306mm</td>
<td>5.52%</td>
</tr>
</tbody>
</table>

Table 8.2 Influence of support removing durations on dynamic response – 7kN/m² UDL

<table>
<thead>
<tr>
<th>Load removal duration</th>
<th>Static deflection</th>
<th>Dynamic deflection-Detailed analysis</th>
<th>Dynamic deflection-Simplified method</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0s</td>
<td>372mm</td>
<td>671mm</td>
<td>722mm</td>
<td>7.60%</td>
</tr>
<tr>
<td>0.2s</td>
<td>372mm</td>
<td>632mm</td>
<td>673mm</td>
<td>6.48%</td>
</tr>
<tr>
<td>0.5s</td>
<td>372mm</td>
<td>543mm</td>
<td>578mm</td>
<td>6.44%</td>
</tr>
<tr>
<td>1.0s</td>
<td>372mm</td>
<td>429mm</td>
<td>452mm</td>
<td>5.36%</td>
</tr>
</tbody>
</table>

8.3.3 Development of TIA framework

With the incorporation of the degraded fire affected floor, the TIA does not require thermal analysis and only requires the nonlinear static response of the floor systems under ambient conditions. Employing the energy-based method, potential dynamic effects along with column buckling can be considered in a simplified, yet reliable manner. For a conservative assessment, the assumption of a sudden column loss due to buckling can be accepted in the TIA in order to predict an upper bound of ductility demand. In this case, the additional strain energy dissipated by the buckled column can be ignored, thus leading to a similar assessment procedure to the typical ductility centred approach for sudden column loss, differing only in the requirement of considering a degraded floor system representing the fire affected floor. In view of this, the proposed TIA framework comprises four main steps (three basic and one optional), namely, nonlinear static floor response, modified nonlinear static response (optional), simplified dynamic assessment, and ductility assessment.

- Nonlinear static floor response

The nonlinear static response of a multi-floor TIA system subject to column removal can be expressed by the total gravity load-deflection response. According to the assumption that the upper ambient floors in conjunction with the degraded floor
(representing the fire affected floor) resist the gravity load in double span, and considering that the ambient floors and the degraded floor have the same predominant deformation mode, the overall nonlinear static response of the TIA system should be taken as the superposition of all the individual floors (degraded and ambient) above the damaged column, as illustrated in Figure 8.16.

![Fig. 8.16 Nonlinear static response of TIA system](image)

The nonlinear static curve can provide a measurement of the energy absorption characteristics of the multi-floor TIA system, which can be expressed as:

$$\delta W = \sum_{j=1}^{n} \delta U_{f,j}$$  \hspace{1cm} (8.6)

where \( j \) represents the floor level, \( \delta U_{f,j} \) is the incremental internal energy absorbed by one individual floor system, for the incremental external work \( \delta W \), the relationship between the gravity load and the incremental system deformation can be given in the following equation:

$$\delta W = \sum_{j=1}^{n} \alpha_j P_j \delta u_{s,j}$$  \hspace{1cm} (8.7)

Due to the compatibility of the deformations of individual floors:

$$\delta u_s = \delta u_{s,1} = \delta u_{s,2} = \ldots$$  \hspace{1cm} (8.8)

and considering the same value of the weighting factor \( \alpha = 0.25 \) for the UDL gravity load distribution, Eq. (8.6) can be expressed by:
\[ \delta W = \alpha \delta u_s \sum_{j=1}^{n} P_j = \alpha \delta u_s P_{\text{total}} = \sum_{j=1}^{n} \delta U_{f,j} \]  

(8.9)

where \( P_{\text{total}} \) is the total gravity load applied on all the floors above the fire affected column. From this equation, the internal strain energy absorbed by all the floors above the fire affected column can be obtained through calculating the area below the \( \alpha P_{\text{total}} - u_s \) curve.

- **Modified nonlinear static response (optional)**

The above nonlinear static response \( \alpha P_{\text{total}} - u_s \) can be directly employed in the third step ‘simplified dynamic assessment’ in order to acquire an upper bound for the ductility demand of the TIA systems by neglecting the strain energy stored in the buckled column. In this case, a maximum deflection of the TIA system subject to an idealised ‘sudden column buckling’ process is obtained. However, in order to reflect a more accurate response of the TIA system subject to localised fire, the \( \alpha P_{\text{total}} - u_s \) curve should be modified with the consideration of the residual column resistance. As discussed before, this is reflected in an additional contribution to the energy distribution:

\[ \delta W = \delta U = \sum_{j=1}^{n} \delta U_{f,j} + \delta U_c \]  

(8.10)

In this expression, the total incremental strain energy \( \delta U \) is comprised of the incremental strain energy dissipated by the floor systems \( \sum_{j=1}^{n} \delta U_{f,j} \) and the incremental strain energy absorbed by the fire affected column \( \delta U_c \) during the buckling process. The overall strain energy absorbed by the column can be obtained through calculating the area under the static response curve of the column under its buckling temperature, as illustrated in Figure 8.17, where \( P_i \) is the maximum load resistance offered by the column under its buckling temperature. In order to predict the buckling temperatures of columns, a simplified numerical method is proposed here which is independent of the axial restraint conditions of the column.
Firstly, a FE model for the considered heated column at an arbitrary temperature free from axial restraint is established. Afterwards, ‘displacement control’ can be applied for the column top vertical displacement as a loading scheme to obtain a static load-displacement response. The column temperature is then adjusted until the peak column resistance is identical to $P_0$, i.e. initial load resisted by the ambient column. This temperature is approximately the column buckling temperature, and the corresponding load-displacement curve is the required curve for calculating the strain energy dissipated by the buckled column under its buckling temperature. This simplified method is based on the fact that one temperature corresponds to only one buckling compressive resistance for a specific column, regardless of its axial restraint conditions; therefore, given a known peak value of buckling compressive resistance $P_i$, which is approximately taken as the ambient value $P_0$, the buckling temperature can be easily estimated using this simplified method.

In order to verify the simplified method, the column buckling temperatures (determined when the column resistance drops abruptly) of the reference building obtained from the previous TDA analysis are compared with those obtained from the simplified method, as listed in Table 8.3. It can be seen that with the availability of numerical tools, the simplified method is capable of providing a simple yet accurate way to predict the buckling temperature of any heated column within a structural system.
Table 8.3 Prediction of column buckling temperature

<table>
<thead>
<tr>
<th>Fire level</th>
<th>Real column buckling temperature - TDA</th>
<th>Column buckling temperature - simplified method</th>
<th>Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>557.5°C</td>
<td>548.2°C</td>
<td>1.67%</td>
</tr>
<tr>
<td>5</td>
<td>585.0°C</td>
<td>581.5°C</td>
<td>0.59%</td>
</tr>
<tr>
<td>8</td>
<td>690.0°C</td>
<td>689.1°C</td>
<td>0.13%</td>
</tr>
</tbody>
</table>

It should be noted that the peak static resistance of an isolated column under its buckling temperature can be realised either slightly above or below the initial position of the column top. Accordingly, the origin of the coordinates which indicates the starting point of column buckling should be shifted to the point of the peak resistance $P_0$, as illustrated in Figure 8.18. In other words, dynamic effects caused by column buckling are considered after the heated column starts to lose its resistance at $P_0$, while the structural response before this critical point is considered as fully static. It is worth noting that the influence of shifting coordinates on the prediction of the maximum ductility demand can be rather limited, because the difference between $u_n$ and $u_o$ is typically negligible compared with the deformation of the considered system.

![Fig. 8.18 Determination of static response of heated column for two alternative cases](image)

Given the predicted column buckling temperature and the corresponding static load-displacement response of the heated column, the total incremental strain energy of the multi-floor TIA system can be obtained by the sum of $\sum_{j=1}^{n} \delta U_{r,j}$ and $\delta U_c$; therefore, the modified nonlinear static response can be taken as the superposition of the two
responses, as illustrated in Figure 8.19, where $u_n$ and $u_o$ are assumed to be identical in this illustration. For the case where $u_n$ and $u_o$ are not identical, the origin of the coordinates of the nonlinear static response curve can be shifted to the point of the peak column buckling compressive resistance $u_n$.

**Fig. 8.19** Method of obtaining modified nonlinear static response of TIA system

- **Simplified dynamic assessment**

Using the modified nonlinear static response of a TIA system under different levels of total gravity load, such as $P_{total,1}$ and $P_{total,2}$, the maximum dynamic response can be obtained from energy balance between the work done by the external load and the internal energy dissipated by the deformed multi-floor TIA substructure and the damaged column. This is illustrated in Figures 8.20(a) and 8.20(b), where $u_{d,i}$ is the maximum dynamic deflection based on the unmodified nonlinear static response considering a sudden column loss process, and $u_{r,i}$ is the reduced dynamic deflection based on the modified nonlinear static responses considering the energy dissipated by the column. Energy balance can be used to determine $u_{r,i}$ as follows:

$$W_i = \alpha P_{total,i} u_{r,i} = U_i = \int_0^{u_{r,i}} \alpha P(\alpha \left\{\frac{U_i}{P}\right\}) d\alpha$$  \(8.11\)

where $U_i$ is the total strain energy absorbed by the floor system and the buckled column, and is equal to the area below the modified nonlinear static curve. Provided that sufficient $\alpha P_{total,i} - u_{r,i}$ points are calculated, a reduced pseudo-static response can be constructed, which depicts the reduced dynamic deflection corresponding to specific values of the gravity load, as shown in Figure 20(c).
Ductility assessment

The last stage of the TIA assessment framework compares the obtained ductility demand (i.e. maximum reduced dynamic deflection) to the ductility supply of the considered TIA system. Based on the Robustness Limit State proposing that collapse of any floor, which can lead to impact loading onto the lower floor, is not permitted, the maximum ductility supply of TIA system should be determined with the avoidance of collapse in any of the affected floors, whilst ensuring that the surrounding columns have sufficient resistance to sustain the redistributed load. According to this definition of ductility supply, system failure occurs when the deformation of either the degraded floor or the upper ambient floors first exceeds their respective ductility capacity. In this respect, the failure of any floor system is attributed to the ductility failure of the first...
surrounding joint on that floor, thus failure criteria are defined in terms of whether the ductility limits of the surrounding joints are exceeded.

Importantly, the current TIA robustness assessment framework assumes a synergetic load resisting action between the degraded floor and the upper ambient floors with an identical deflection. This ignores the potential of punching shear caused by the shear failure of fire affected steel beam-to-column connections, as illustrated in Figure 8.21(a). To address this, two supplementary solutions can be employed to evaluate the risk of punching shear. Firstly, punching shear checking can be treated in a conventional way that directly compares the overall shear capacity of the steel connections at elevated temperatures to the design shear force they resisted, provided that the maximum temperature is known. Alternatively, if the maximum temperature is unknown, which is a more likely situation in line with unforeseen extreme events, it can be conservatively assumed that the shear resistance of the affected connections are abruptly removed, thus a simplified TIA method can be applied to the individual detached degraded floor, as illustrated in Figure 21(b). The maximum deflection/ductility demand of the TIA detached floor can be obtained in a similar way to the TIA considering the top floor under fire, but assuming a sudden shear failure response without considering the extra strain energy dissipated by column. Therefore, this event-independent method can be deemed as a supplementary step for the TIA framework to assess the robustness of structures subject to unforeseen shear failure of fire affected steel connections, regardless of the status of the fire affected column.

*Fig. 8.21* TIA assessment of detached floor due to shear failure of fire affected connections
8.4 Illustrative examples for application of TIA

This section illustrates the application of the proposed TIA, where the reference car park considered previously using the TDA is employed (as reproduced in Figure 8.22). Progressive collapse assessments are performed at the same three affected floor levels as considered in the TDA, i.e. floor levels 1, 5, and 8. Full 2D slab models are employed in the current TIA framework, while the lower idealisation level based on a grillage model is not adopted in this study as it does not incorporate 2D slab effect. The value of the unfactored basic gravity load is taken as 5kN/m² (2.5kN/m² dead + 2.5kN/m² live), which leads to the value of $\alpha P$ equal to $0.25 \times 3200\text{kN}=800\text{kN}$ for a single floor.

![Reference car park for TIA assessment](image)

The nonlinear static curves are obtained using the FE models established in ADAPTIC for the degraded floor as well the ambient floor (as shown in Figure 8.23) and subsequently assembling these for the overall response, as shown in Figure 8.24. The degraded floor model with $0H$ removed beam length is employed to represent the fire affected floor in a TIA manner. The ductility supplies for the individual ambient floor and the degraded fire affected floor are 610mm and 692mm in deflection, respectively, where failure modes for both floor systems are governed by rupture of rebars in the surrounding major axis beam-to-column joints under hogging moments. By means of superposition, unmodified nonlinear static responses $\alpha P_{\text{total}} - u_0$, for floor levels 1, 5, and
8 are obtained, where for the TIA systems with floor levels 1 and 5 subject to fire, the ductility supplies are governed by the deflection capacity of the upper ambient floors.

![Graph](image1)

**Fig. 8.23** Nonlinear static response of individual floor

![Graph](image2)

**Fig. 8.24** Unmodified nonlinear static response of multi-floor TIA system

The nonlinear static curves in Figure 8.24 can be modified to additionally consider the strain energy dissipated by the fire affected columns, leading to a reduced dynamic effect. Using the simplified method discussed in Section 8.3.3, the nonlinear static response of the internal columns HEB 550, HEB 300 and HEB 200 designed for floor levels 1, 5, and 8, respectively, are obtained as shown Figure 8.25, enabling the determination of the column contribution to strain energy. It can be observed that significant vertical load resistance can be still maintained until a column top vertical
displacement of 25mm; afterwards, the static column resistance starts to drop quickly until a column top vertical displacement of around 50mm, beyond which the residual column resistance is negligible. In view of this, assuming a sudden loss of fire affected columns without considering the residual resistance can lead to a significant overestimation of the corresponding ductility demand. Furthermore, the peak compressive resistances of the columns are achieved slightly above or below the initial ambient column top position (typically within 10mm), so the origin of the coordinates which indicates the starting point of column buckling should be shifted to these positions in the modified nonlinear static curves.

Combining the obtained nonlinear static response of the TIA floor systems and the deflected columns, modified nonlinear static curves can be obtained, as shown in Figures 8.26 to 8.28 for fire at floor levels 1, 5 and 8, respectively. Based on the principle of energy equivalence, the ductility demands (reduced dynamic deflections) of the reference car park subject to localised fires at floor levels 1, 5, and 8 are obtained as 444.6mm, 479.3mm and 709.9mm, respectively. These results demonstrate increasing ductility demands with the floor level affected by fire, which is due to greater resistance provided by the upper ambient floors when the fire occurs at lower floor levels. Furthermore, it can be seen that the influence of shifting coordinates on the predicted maximum ductility demand is indeed negligible due to the small difference between \(u_n\) and \(u_o\) compared with the maximum deflections. Therefore, the procedure of coordinate shifting can be neglected in practical assessment, thus the starting point of the modified
nonlinear static curve can be at the initial ambient column top position. Comparing to the ductility supplies, the ductility demands of the structure subject to fires at floor levels 1 and 5 are safely accommodated, which indicates sufficient robustness. On the other hand, for fire at the top floor (level 8), the ductility supply is exceeded by the ductility demand, so a high potential for progressive collapse is indicated for the structure. Table 8.4 provides the ductility supplies and demands of the structure subject to the three fire affected floors. For comparison purposes, sudden column loss responses which are obtained through the unmodified nonlinear static curves are also given, where larger ductility demands are predicted.

Fig. 8.26 Ductility supply and demand of structure subject to fire at floor level 1

Fig. 8.27 Ductility supply and demand of structure subject to fire at floor level 5
From the perspective of robustness, capacity/demand ratios (CDR) are also used here to indicate the potential of the structure for progressive collapse, which are expressed in terms of the structural resistance at the point of failure compared to the applied loading. Unlike the assessment of CDR in the TDA, the considered CDR for the TIA is independent of temperature. Table 8.5 provides the CDR’s using the TIA for the structure subject to fires at floor levels 1, 5 and 8 with a basic gravity load of 5kN/m$^2$. The gravity load capacities are obtained using the modified nonlinear static curves, such that the reduced dynamic deflection is identical to the ductility supply.

**Table 8.5 CDR’s for reference car park using the TIA**

<table>
<thead>
<tr>
<th>Fire floor level</th>
<th>Gravity load capacity (kN/m$^2$)</th>
<th>Gravity load applied (kN/m$^2$)</th>
<th>CDR (Capacity / demand ratio)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.23</td>
<td>5</td>
<td>1.246</td>
</tr>
<tr>
<td>5</td>
<td>5.96</td>
<td>5</td>
<td>1.192</td>
</tr>
<tr>
<td>8</td>
<td>4.86</td>
<td>5</td>
<td>0.972</td>
</tr>
</tbody>
</table>
The CDR’s predicted by the simplified TDA are utilised to compare with those obtained by the TIA in order to verify the reliability of TIA robustness predictions. Since the CDR’s obtained by the TIA, as given in Table 8.5, are based on the robustness of the structure after column buckling but before punching shear, the CDR’s obtained by the simplified TDA under a maximum temperature of 750°C are employed, where the condition column buckling mainly governs the collapse mode. Table 8.6 provides the CDR’s predicted by both TDA and TIA models.

<table>
<thead>
<tr>
<th>Fire floor level</th>
<th>CDR predicted by TDA (max. temperature 750°C)</th>
<th>CDR predicted by TIA</th>
<th>Discrepancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.400</td>
<td>1.246</td>
<td>11.0%</td>
</tr>
<tr>
<td>5</td>
<td>1.390</td>
<td>1.192</td>
<td>14.2%</td>
</tr>
<tr>
<td>8</td>
<td>1.134</td>
<td>0.972</td>
<td>14.3%</td>
</tr>
</tbody>
</table>

It can be seen that the CDR’s predicted by the TIA are relatively conservative, and this can be attributed to three reasons. Firstly, the strain energy absorbed by the entire TIA system may be underestimated due to the idealisation of the degraded floor system, where the strain energy dissipated by the removed floor components (e.g. fire affected joint) is ignored. Secondly, more energy may be dissipated via compressive arching effects of the fire affected floor along with column buckling, which is underestimated by the ambient degraded floor system. Finally, as indicated in Chapter 7, thermal expansion can be beneficial in mitigating the ductility demand, so ignoring it in the degraded floor system may lead to conservative predictions. Despite these inaccuracies, the discrepancies of CDR between the TDA and TIA predictions are within 15% on the conservative side, which is acceptable for design application.

Considering possible punching shear failure of the TIA model, the supplementary solution discussed in Section 8.3.3 can be employed, where the ductility demand of the TIA detached floor can be obtained using a similar approach as that of the TIA system considering the top floor under fire, but assuming a sudden shear failure response instead. This method is conservative, where the corresponding CDR of the structure using the TIA for fire at any floor level is only calculated as 0.968, which is much lower than that predicted by the TDA model considering a shear-governed collapse mode under the maximum temperature of 1000°C (1.318 and 1.352 of CDR for fire at floor...
level 1 and 5). This can be explained by the fact that punching shear normally occurs after column buckling according to the simplified TDA predictions. Therefore, before punching shear, a considerable floor deflection is already developed due to column buckling but is subsequently stabilised due to the ‘pulling-up’ effect provided by the upper ambient floors. Therefore, the assumption in the TIA framework that punching shear occurs at the initial column top position inevitably overestimates the dynamic deflection. However, if a reduced temperature near the column base is considered or the fire affected column is protected to allow the occurrence of punching shear before column buckling, the assessment of punching shear in the TIA framework can still be reliable.

8.5 Concluding remarks

A TIA framework is developed in this chapter for the practical design-oriented robustness assessment of multi-storey steel/composite structures against localised fire that is event-independent. A novel characteristic of the TIA is its ability to evaluate the dynamic effects associated with column buckling through a simplified energy balance approach, based on which structural robustness is assessed through four steps. The first step ‘nonlinear static floor response’ is aimed at obtaining the energy absorbed in the floor system as the area under the static load-deflection curve, where the fire affected floor is ideally represented by an ambient degraded floor with direct removal of several fire affected components. This is followed by an optional second step ‘modified nonlinear static response’ which considers the additional strain energy dissipated by the fire affected column during buckling, thus potentially allowing a reduced dynamic deflection compared with that caused by sudden column loss. The amount of energy dissipated by the fire affected column is equal to the area below its static load-displacement curve under the buckling temperature, where this temperature is obtained through a simplified numerical method. Acquiring the total energy dissipated by the TIA system, consisting of the floor systems and the fire affected column, the maximum ductility is obtained via the third step ‘simplified dynamic assessment’ using the principle of energy balance. Based on this principle, the maximum floor deflection is achieved when the external work done by the gravity load equals the total energy dissipated by the TIA system. Finally, in the fourth step ‘ductility assessment’, the
ductility supply of the TIA floor system is compared with the obtained ductility demand, and structural robustness is evaluated accordingly.

To illustrate the application of the proposed TIA, the reference car park investigated in the previous chapter using the TDA is considered using the TIA, where the same fire locations are assumed. Full slab models are employed to consider 2D membrane action which has been shown to be beneficial in improving the loading resistance of double-span floor systems against column loss. The results show that in terms of the TIA assessment framework, the reference building generally has a sound robustness against localised fire, although the CDR for the top floor under fire is marginal. Comparing with the TDA results, the TIA predicts a reliable structural robustness response on the conservative side, provided that progressive collapse is triggered by column buckling. As noted from the TDA, however, punching shear can occur at around 200°C-300°C higher temperature than the column buckling temperature. The punching shear failure type has been considered in the current TIA framework, but it is treated in a simplified way that may lead to an overly conservative result.

As a general remark on its applicability, the TIA offers for the first time a rational event-insensitive design-oriented framework that deals with dynamic effects and ductility considerations in progressive collapse assessment for structures under localised fire. Moreover, similar to the sudden column loss scenario that has been adopted in major design codes for robustness assessment of structures subject to extreme dynamic loading, the local damage scenario proposed for the TIA has a significant potential for codification to deal with localised fire.
CHAPTER 9

Summary and Conclusions

9.1 Summary

The term ‘robustness’ is primarily associated with the ability of a structure to resist progressive collapse, which is a catastrophic type of failure initiated by severe local damages of structural members. This failure type is not typically considered in standard design practice, but more effort has been devoted to this issue recently, as driven by a number of catastrophic building collapses reported during the last four decades. Conventional robustness assessment is mainly associated with blast and impact loading, though with the development of building design concepts based on the utilisation of steel components, more comprehensive investigations on the robustness of unprotected steel-composite structures under elevated temperature are required. However, no systematic design framework is currently available which is sufficiently practical and rational to incorporate fire into structural robustness design, thus bringing difficulties for structural engineers to perform an effective yet economical robustness design for localised fire. In this context, the various robustness assessment frameworks introduced in this thesis are aimed at bridging the gap between the current codified treatments of fire hazards and progressive collapse, where localised fire that are typically caused by burning vehicles in multi-storey steel-composite car parks are considered as a main reference scenario.

Current design approaches used for mitigating the risk of progressive collapse are mainly categorised under two types, namely, direct and indirect methods. In direct design, local damage of key structural members needs to be identified, and for
conservative purposes these affected members are normally considered as ‘completely lost’. Subsequently, detailed analysis is performed to check if the available ductility supply in the remaining undamaged structure is adequate to accommodate the extent of damage. On the other hand, in indirect design, general prescriptive measures are utilised to enhance the robustness without the need of detailed performance-based structural analysis under specific extreme loading events. Each method has its own advantages and can be implemented in different design stages to fulfil various design requirements.

The robustness assessment methods developed in this thesis is comprised of four basic components, namely, detailed Temperature-Dependent Approach, simplified Temperature-Dependent Approach, Temperature-Independent Approach, and practical design recommendations. The four components have been comprehensively discussed in Chapters 5, 6, 8 and 7 respectively. This framework incorporates both of the concepts of direct and indirect design methods, as demonstrated in Figure 9.1. In line with the concept of direct design methods, the detailed TDA, the simplified TDA, and the TIA rationally evaluate the ability of structures to withstand the localised fire in a performance-based manner, where different levels of event-sensitivity are considered for the three approaches. On the other hand, practical design recommendations are also developed along the lines of prescriptive design guidance. The related conclusions drawn from this method can be employed in future for indirect design guidance by engineers to make preliminary decisions on robustness under localised fire.

![Robustness Assessment Framework](image)

**Fig. 9.1** Developed robustness assessment framework

The development of the four approaches is based on extensive numerical studies performed using the advanced nonlinear finite element program ADAPTIC (Izzuddin, 1991). Therefore, the reliability of the underlying principle of these approaches is
evidently associated with the accuracy of the corresponding numerical modelling technique. In order to ensure the accuracy of the structural model, benchmark studies on individual members, e.g. beams, columns, slabs, and joints, are conducted first in Chapters 3 and 4, where various experiments as well as numerical studies conducted by other researchers are selected as benchmarks. Considering the fact that the ductility demands of structures following initial local failure are usually concentrated in the joint regions, predefined joint failure criteria form the basis of the overall structural failure criteria. A detailed discussion on the joint behaviour under extreme loading is presented in Chapter 4. In the following sections, conclusions summarised from the study of joints are underlined first. This is followed by a summary of the applications of the robustness assessment framework. In recognition of the potential shortcomings for each approach in this framework, possible further enhancements are identified in the last section for future research.

9.2 Joint ductility supply and Robustness Limit State

This study proposed a system failure criterion based on the ductility characteristics of joints. With respect to the available joint modelling techniques, it is proposed that compared with the other options (e.g. curve fitting approach, sophisticated FE approach, and experimental approach) the component-based models can have sufficient accuracy and practicality for application at an intermediate level of sophistication. Importantly, the component-based modelling technique enables the global frame model to directly incorporate the joint models (comprising spring series) with the consideration of axial joint deformability, such that the interaction between bending moment and axial force can be captured accurately. The component joint model developed in this study remedies the deficiency of the single rotational spring model used to represent the joint behaviour as specified in Eurocode 3 (2005a). The rotational spring model can only predict the flexural bending characteristic of the joint, and it neglects the axial joint deformability. Furthermore, when elevated temperature conditions are considered, degraded joint performance can be simulated in the component joint model through degrading the properties of the springs. In particular, temperature variations (temperature gradients) within the joint can be easily reflected by degrading the different parts (different springs) of the joint model at various rates.
The general robustness assessment procedure based on the response of joints is illustrated in Figure 9.2, where components are the most basic and important units that constitute the entire joint component model. Chapter 4 provided a ‘component library’ from which activated components can be selected for various joint types. For each component, four main characteristics were identified as essential in determining the overall joint response, namely, elastic resistance, elastic stiffness, post-elastic stiffness, and maximum deformation. Among the four characteristics, elastic resistance and elastic stiffness are two basic parameters mainly used for obtaining the elastic bending resistance and the elastic rotational stiffness of the considered joints, and they have already been extensively studied and stipulated in Eurocode 3 (2005a). Another key issue of deducing a realistic joint ductility performance is to accurately estimate the post-limit parameters, including the post-limit (post-elastic) stiffness and maximum deformation. An evident knowledge gap still, however, exists in this area. This thesis presented relevant experimental data and other numerical and analytical research results which provide helpful information towards the proposal of multiple joint failure criteria. An important conclusion with regard to the post-limit characteristics of components is that all the commonly used components can be potentially categorised into three main types, namely, ductile component, limited-ductility component, and brittle component. It was also suggested that failure of brittle component should not govern the failure mode in order to ensure sufficient joint ductility supply. The post-limit stiffness (strain hardening) of ductile and limit-ductile components was found to have significant
influence on the inelastic rotational stiffness and the ultimate bending capacity of the joints. Through comparisons with test data, it was preliminarily concluded that employing ductile and limited-ductility components in a joint model with a strain hardening coefficient of 1% to 3% can lead to a favourable prediction.

When joint component models are exposed to fire, the degradation of joint resistance and stiffness is simulated in this study with desirable reliability. The main methodology considering elevated temperature effects is to predefine a performance degradation vs. temperature curve (strength reduction and stiffness reduction curves) for the spring models representing the components, such that the characteristics of the joint models can be degraded during structural analysis, where the temperature information is input as part of the load case. In order to verify the accuracy of the component joint model under elevated temperature, various joint fire tests were selected. It was found that the elevated temperature behaviour of the joints predicted by the joint component models compares well with the test results within the elastic range (i.e. initial bending stiffness, elastic bending resistance).

Considering the overall joint ductility supply, which is one of the most important characteristics for evaluating structural robustness, multiple joint failure criteria were proposed in this study, where three possible failure scenarios were identified in addition to the previously established first failure of bare-steel joint components, namely, overall deformation limit of bolt-rows, rupture of rebars, and joint shear failure. It was proposed that a joint is deemed to fail when any of the four failure scenarios is first met. The reason for setting an overall deformation limit of bolt-rows is due to the unrealistic assumption of an infinite deformation capacity of the ductile components. Based on available test data, the maximum bolt-row elongation is defined as 25mm for end-plate joints and 35mm for angle or fin-plate joints in the current joint model, regardless of temperature. However, it was found that the proposed maximum bolt-row elongation is always conservative, particularly under elevated temperature conditions. Recent joint tests in the ROBUSTFIRE project (Fang et al., 2011) also showed that the tested joints can maintain a considerable bending resistance with the bolt-row elongation of more than 50mm under the temperature of around 500ºC. In view of this, the limit of the maximum bolt-row elongation in the current joint failure criteria may need to be relaxed under elevated temperature in practical application, subject to further experimental
evidence. When joints are designed in terms of composite behaviour, rupture of rebars was also assumed as a possible governing failure mode, where the method proposed by Anderson et al. (2000) is employed to calculate the maximum elongation of the rebars. Finally, shear failure can occur when the shear capacity of the joint is exceeded. In the component model, joint shear failure is simulated by defining the maximum shear capacity of the component spring representing the joint shear behaviour which is uncoupled from the bending/axial joint response.

Based on the failure criteria of joints, a Robustness Limit State (RLS), which is similar to the concepts of the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS), was adopted from earlier work at Imperial College London (Izzuddin et al., 2008, Izzuddin, 2009) to provide the failure criterion for overall structural systems. The RLS considered in this study allows no floor collapse subsequent to column loss led by localised fire, according to a relevant study (Vlassis et al., 2009) which indicated that floor failure can have a great potential to trigger progressive collapse for typical steel-composite building structures. Floor collapse was assumed in the present work to occur upon failure of any joint along the remote perimeter of the area surrounding the affected column, but first failure of the fire affected joints near the buckled column is not considered by itself to lead to floor collapse. As a general remark, the current RLS treats the issue of progressive collapse in a conservative way because it ignores the potential residual resistance of the surrounding joints after the rupture of the rebars or even after the rupture of one or two bolt-rows, considering the fact that moderate tensile capacity can still be maintained in the joints when catenary action of the beam is formed. Additionally, slab membrane action can contribute to the progressive collapse resistance regardless of the condition of the surrounding joints, as long as sufficient supports are present along the edges. However, these safety margins have not been convincingly supported by relevant research, so they were not considered in the current failure criteria. Of course, when more convincing data are available, the failure criteria proposed in this study can be easily modified accordingly.

### 9.3 Robustness assessment framework

Incorporating the proposed failure criteria, a robustness assessment framework with various levels of sophistication was developed to cater for different design requirements.
In line with the *direct design approach* and the *indirect design approach* recommended in current design codes for progressive collapse assessment against blast, the approaches considered for localised fire can be also categorised into these two types. The application of the approaches was illustrated, where a reference car park structure was presented and the corresponding system models were developed. Considering the model reduction technique that can simplify the assessment procedure, as previously proposed by Izzuddin *et al.* (2008), the system model was developed in a multi-level manner. For all the levels of the system model, the influence of the surrounding indirectly affected structures was idealised by linear boundary springs applied at the ends of the beams, where the values of the stiffness were taken as the axial stiffness of the adjacent beams and joints. For the reduced system model with the floor slab presented using either 2D elements or a grillage approximation, the upper ambient floors were modelled as a single nonlinear spring applied at the top of the fire affected floor, and this modelling approach was shown to provide favourable accuracy.

### 9.3.1 Direct design approach

The direct design approach is normally performance-based and relies on sophisticated numerical tools. Under localised fire conditions, it requires a series of analyses based on the selected fire scenarios and then assesses the robustness of the structure based on the predefined failure criteria. Three sub-approaches were presented in this study under the direct design approach, namely, Detailed Temperature-Dependent Approach, Simplified Temperature-Dependent Approach, and Temperature-Independent Approach.

*Detailed Temperature-Dependent Approach*

Compared with the other approaches, the detailed TDA is the most sophisticated yet computationally expensive approach under the current robustness assessment framework, so it is more suitable for research and application to important safety-critical structures. In this approach, details of the fire scenario, e.g. position and heat release rate of the fire, need to be considered and thermal analysis is required in order to obtain the actual temperature distribution over the time domain within the structural model. Given the temperature distribution as an input, dynamic structural analysis is conducted to capture the realistic robustness behaviour of the structure. The structural analysis is usually performed over a time domain in this approach, so the fire resistance time can be estimated accordingly. In Chapter 5, the application of the detailed TDA
was illustrated for the reference car park, where the floor slab was modelled using either 2D shell elements or beam-column elements considering a grillage approximation. It was concluded that the grillage model which is usually employed for conventional structural designs under normal loading is too conservative. On the other hand, the full slab model, which indicated much lower potential for progressive collapse of the reference car park, is more suitable for structural robustness assessment that is associated with extreme loading and large deflections.

As predicted using the detailed TDA, three types of first joint failure modes were observed in the reference building subject to the selected fire scenario, namely, single-span failure type, double-span failure type and shear failure type (punching shear). Among the three failure types, only the latter two are commonly found which are associated with column buckling and joint shear failure at elevated temperature, while the single-span failure type due to the failure of the fire affected joint under normal hogging moment before column buckling is less likely to occur (it is only found in the grillage model). Although first joint failure does not necessarily mean an immediate system failure, precautions may still be required to strengthen the joints that are more prone to failure, bearing in mind the main aim of reducing the potential of progressive collapse and also reducing repair work after fire. Some of the identified joint failure types can be effectively avoided from a construction perspective. Take the shear failure type for example, apart from the conventional strategy of applying fire protection, a simple and cost-effective way which utilises the shear resistance of the slab by passing the rebars through small openings in the column flange/web may be effective in mitigating the risk of punching shear. Of course, this strategy needs further investigation and experimental validation.

_Simplified Temperature-Dependent Approach_

Following the discussion of the detailed TDA, parametric studies were conducted in Chapter 6 with the aim of identifying the primary thermal characteristics that influence the overall robustness of structures under localised fire. It was observed that the details of the temperature distribution within the directly affected zone have a less significant influence compared with the contribution from the surrounding ambient structural members, including the ambient joints and slab. Recognising the complexity of fire modelling and thermal analysis, and considering the fact that some thermal
characteristics during a localised fire have insignificant influence on overall structural robustness, the detailed TDA can be simplified. The simplified TDA only considers the position, the range, and the maximum temperature of the localised fire; therefore, it is more close to typical robustness provisions, which are intended to limit the progression of local damage under unforeseen loading as event-insensitive as possible.

The main methodology of the simplified TDA is to develop a monotonically increased ‘block’ temperature model that can be directly applied onto the structural model to avoid complex heat transfer analysis, enabling structural analysis to be performed over a more event-insensitive temperature domain (instead of the time domain). In this study, a simplified temperature distribution (i.e. uniform temperature along the member length, and linear temperature distribution over the cross-section) was proposed within the rectangular fire affected range. Through comparisons with the predictions from the detailed TDA, the simplified TDA was shown to provide a reliable yet more efficient solution. The discrepancies of the predicted key structural response parameters, including column buckling temperature, first joint failure temperature, first failure mode and deflection, were typically within 5%.

Furthermore, a study was performed under the simplified TDA framework to investigate the significance of dynamic effects along with column buckling, where the actual floor deflection after column buckling was compared with two limiting conditions, namely, static column loss and sudden column loss. It was concluded that dynamic effects arise during column buckling, and that the actual deflection is always between the two extreme deflections. In addition, dynamic effects tend to become more significant with increasing gravity load and with fire at higher floor levels.

Finally, the capacity-demand ratio (CDR), which is a typical robustness indicator, was discussed in this study to extend its use to localised fire conditions. Differing from conventional robustness assessment, the CDR considered under fire clearly depends on temperature. In this context, a classification of fire severity was proposed in this study, namely, basic/frequent fire, intermediate fire, and severe/rare fire, corresponding to the maximum temperatures of 500°C, 750°C and 1000°C, respectively. This classification is inspired by the probability assessment strategy for seismic loading, where different levels of seismic loading are considered for different seismic intensities. Under the simplified TDA analysis which is performed over the temperature domain, the CDR can
be easily obtained through setting the maximum temperature and comparing the maximum load capacity with the actual applied load at that temperature. For the considered reference structure under the basic load of 5kN/m\(^2\), it was found that the CDR is always larger than 1.0 up to the maximum temperature of 1000\(^\circ\)C, thus indicating a good robustness of the car park.

It is worth noting that the main shortcoming of the simplified TDA is its inability to predicting the fire resistance time. This problem can be potentially solved in future by proposing predefined temperature-time curves similar to the standard ISO fire curve for typical car parks, but additionally considering the input parameters, such as the number, location, and heat release rate of the burning vehicles, as well as ventilation conditions. Through this strategy, a codified chart, either semi-empirical or calculation-based, may be available to roughly predict the temperature-time variation during a localised fire, thus the maximum fire resistance time of the structure can be derived from the maximum temperature that is predicted by the simplified TDA.

*Temperature-Independent Approach*

As a further simplification of the robustness assessment framework, the TIA was developed towards a fully event-independent strategy for design-oriented application, in the sense that the maximum temperature is assumed to be unknown. Based on the fact that certain parts of structural members under high temperatures can considerably lose their strength, thus offering no significant contribution to progressive collapse resistance, these structural members are considered as completely removed within the TIA framework. Therefore, the TIA model does not require thermal analysis, and can provide a simplified robustness assessment procedure regardless of temperature. This strategy is inspired by the idea of the event-independent ‘sudden column loss’ scenario currently adopted in some of the commonly used design codes for progressive collapse assessment (GSA, 2003; DoD, 2009). In the current TIA framework, the fire affected joint, column and parts of the beams are considered as removed, where it was found that the length of the beam removed has negligible influence on the overall structural robustness.

Moreover, the TIA does not require dynamic analysis; instead, static analysis using the principle of energy balance is employed to estimate the dynamic response during
column buckling. This is an extension of the previous approach (Izzuddin et al., 2008, Vlassis et al., 2008) developed at Imperial College London for assessing the robustness of steel/composite buildings subject to sudden column loss scenarios, where the TIA additionally considers the residual column resistance that can potentially reduce the magnitude of dynamic effects. The basic methodology of the TIA is to balance the external work done by the gravity load with the strain energy absorbed by the floor systems (degraded and ambient floors) as well as the buckled column. Based on this methodology, four steps are involved in the TIA framework, namely, *nonlinear static response*, *modified nonlinear static response*, *simplified dynamic analysis*, and *ductility assessment*. The first step is to obtain the superposition of the nonlinear static force-deformation relationships of all the affected floors above the fire affected column, and the area below the static force-deformation curve is equivalent to the energy absorbed by the floor system. The second step is to modify the static force-deformation curve by considering the additional energy dissipated by the buckled column. In the third step, dynamic floor deflection is obtained using energy balance, and finally in the fourth step the dynamic deflection is compared with the ductility supply to establish the proximity to the Robustness Limit State.

Employing the TIA, the robustness of the reference car park considered under the TDA framework was reassessed, and it was observed that the TIA predictions correlate well with the TDA results, although TIA predictions are always on the conservative side. In order to evaluate the risk of progressive collapse due to punching shear under a severe fire, a supplementary step was proposed with the conservative assumption that punching shear always occurs abruptly from zero floor deflection before column buckling. Evidently, the supplementary step leads to overly conservative predictions with a considerable safety margin. However, as mentioned previously, the risk of punching shear can be eliminated with appropriate construction measures, in which case punching shear checking may be unnecessary in the TIA.

As a summary, the TIA offers for the first time a rational event-independent design-oriented framework that simultaneously deals with fire conditions, dynamic effects and ductility considerations in progressive collapse assessment in a performance-based manner, enabling engineers to quickly check the structural robustness and to make corresponding design decisions regardless of temperature.
9.3.2 Indirect design approach

The indirect design approach focuses on increasing the robustness of structures through prescriptive rules, which enable more efficient preliminary design. The indirect design approach for conventional robustness assessment considering blast loading is well presented in ODPM (2004), where the ‘tie force method’ is popular in the UK and has been assumed to offer a minimum level of robustness. In addition, structural type selection was also discussed by various researchers seeking effective prescriptive solutions. A similar indirect design approach can be employed for localised fire conditions, where emphasis may be additionally given to fire protection schemes and even architectural issues.

With this aim, this study investigated different structural design schemes via parametric studies, thus gaining understanding of the influences of the considered parameters on structural robustness from a design perspective. The parametric study was conducted on the eight-storey composite car park considered under the simplified TDA framework, where the different car park designs were considered under the same idealised monotonically increased temperature. The factors investigated in this study which potentially influence the structural robustness against fire include: influence of fire protection on the steel beam, influence of fire protection on the steel joint, influence of fire protection on the fire affected column, influence of fire protection on the slab metal deck, influence of column stiffener, influence of joint rebar, influence of slab reinforcement mesh, and influence of slab metal deck thickness. Through the outcomes from the parametric study, a prescriptive robustness design recommendation for typical steel-composite structural buildings against localised fire was proposed. It was mainly concluded that applying fire protection for increasing structural robustness under localised fire is neither cost-effective nor necessary, while enhancing the ambient response of joints and slabs can greatly enhance the robustness. Towards possible design solutions, it was further concluded that several design schemes or scheme combinations can be effective in improving structural robustness under localised fire, regardless of the considered fire floor levels, maximum temperature, and modelling assumptions. The effective schemes or scheme combinations are: 1) applying column fire protection with joint fire protection, 2) increasing the amount of joint reinforcement while employing a column web stiffener, 3) increasing the amount of slab mesh, and 4)
increasing the steel deck thickness. The selection of these schemes depends on actual design requirements, where cost is an important factor.

### 9.4 Future research

The robustness assessment framework proposed in this thesis sheds considerable light on design methodologies for structural robustness under localised fire. However, there is clearly scope for further improvement towards the provision of more realistic and effective design approaches, and future research is needed to broaden the applicability of the proposed methods. Potential future research topics include the following:

- **Realistic prediction of ductility supply.** Joint ductility supply is one of the most essential aspects for realistic assessment of structural robustness. It is recognised that the joint failure criteria considered in this study may be conservative in view of recent experimental evidence provided by the ROBUSTFIRE project. Towards more realistic system failure criteria, a large-scale experimental programme to establish a more clear understanding of the residual floor resistance after first joint failure (e.g. rupture of rebars or even several bolt-rows) as well as the successive joint failures can be beneficial.

- **Simplified design-oriented system models.** A detailed discussion on the multi-level system model has been presented in this study, where it was concluded that the reduced full slab model (i.e. Level B model) can provide a favourable prediction. However, establishing a full slab model is still demanding in terms of modelling complexity, so there is a need for the development of simplified structural models that can facilitate the application of the framework in design without a significant compromise of accuracy. A possible solution in future is to establish an assembly of beam models taking account of nonlinear joint response and composite action. The actual load resistance of the beam assembly can be derived from either hand calculation or numerical analysis, multiplied by an enhancement factor to allow for the 2D slab membrane action.

- **Other fire scenarios.** Emphasis of this study was placed on structural robustness subject to localised fire affecting internal columns. In order to enhance the application of the current framework, further studies on fires at external columns (i.e. façade columns and corner columns) are required. In addition, possible
multiple column buckling scenarios need to be considered, particularly when the column spacing is small and the range of fire is large. In spite of the various possible fire scenarios, the currently proposed robustness assessment framework is still applicable and can be readily implemented with small amendments if necessary.

- **Realistic boundary conditions.** It was assumed in the current structural system model that the boundary restraints from the adjacent unaffected structure are represented by linear springs, where the stiffness was ideally calculated from the axial stiffness of the adjacent beams and joints. Further improvement of the development of the boundary springs may include two aspects. The first one is to consider the diaphragm effects of the adjacent slab as well as the shear stiffness of the shear studs which can influence the boundary restraints. The second one is to consider the coupling effect of the boundary restraints among floor levels, although slab diaphragm action may considerably reduce this effect.

- **Prescriptive rules with 'code-format'.** At the current stage, some of the prescriptive rules (e.g. influences of rebars, mesh reinforcement, and steel deck) only highlight the potential key parameters affecting structural robustness under fire, while no further details are provided on the quantification of member design in specific. In future investigations, practical requirements for designing structural members, including stipulating the minimum reinforcement ratio/steel deck thickness of slabs, may be feasible in design codes.

- **Combined effects of blast and fire.** Fire is usually triggered by various scales of blast. The influence of an initial local damage state caused by blast before the development of fire may need to be considered, particularly when the blast is significant enough to cause significant damage of the affected structural member. Major concern should be given to joint damage and column deformation, because these two aspects are closely related to the predictions of joint ductility supply and column loss temperature, respectively.

- **Architectural-based fire strategies.** The consideration of structural robustness under localised fire from an architectural perspective was beyond the scope of this study, but it can be as effective as that from the structural perspective. Potential treatments include the following aspects: planar layout, floor height,
fire isolation barriers and doors, and new construction materials. These strategies deserve further investigation.
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