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Executive Summary

Progressive collapse of a building can be regarded as the situation where local failure of a primary structural component leads to the collapse of adjoining members and to an overall damage which is disproportionate to the initial cause.

The problem of progressive collapse in civil engineering gained interest in 1968, after the partial collapse of the Ronan Point apartment building in London UK. Following this event, intensive research effort led to developing progressive collapse strategies and methods and resulted in the first progressive collapse provisions in the UK standards. Even current methods and strategies benefit to a great extent from the approaches developed at that time. A second and third wave of progressive collapse interest by the civil engineering community appeared after the disproportionate collapse of the A.P. Murrah Federal Building (Oklahoma City, 1995) and the total collapse of the World Trade Center towers, both caused by terrorist attacks.

Progressive collapse can be triggered by many different actions. Examples of such actions can be: explosions caused by gas or explosives; impacts of vehicles, ships or planes; earthquakes; human errors in the design or construction phase etc. The prediction of such events and abnormal actions is very difficult and depends on many factors. From the security point of view, progressive collapse is of particular importance as buildings and other critical infrastructures often become the target of terrorist bombing attacks. A structure should be capable to suffer local damage but to prevent excessive spreading of it to other members. However, designing a large building against progressive collapse due to blast loading is quite a challenge because of the several assumptions and unknown parameters involved: the quantity and type of the explosive charge, the distance from the building where the explosive is detonated, whether the blast affects corner or central columns of the building etc. These difficulties make that there are effectively no provisions in the national construction codes and standards for the design of structures to resist external explosions or internal explosions caused by explosives.

This report presents several definitions and proposals of robustness measures of structures, and it provides a review of procedures and strategies for progressive collapse design based on selected codes, standards and guidelines mainly from the EU and USA. As shown, the early developed design approaches for progressive collapse mitigation are divided into indirect and direct ones. Indirect approaches consist of applying prescriptive design rules (minimum requirements on strength, continuity, ductility, redundancy), contributing to the resistance to progressive collapse. However progressive collapse behaviour is not addressed explicitly. These indirect design approaches address the problem by identifying and incorporating into the building system characteristics that enhance robustness, without special consideration to loads or events that could trigger disproportionate collapse. On the other hand, direct approaches involve a performance-based design and consist of the specific local resistance method (the design of some "key elements" to resist a sufficiently high pressure) and the alternate load path method.

The report also includes some real cases of progressive collapses, it provides a representative view of research efforts in the field, as reported in international journals and conferences, and points out knowledge gaps.

Contents

1. Introduction

This is an updated version of the previous report on literature survey in robustness and progressive collapse of structures, which can be considered as the situation where local failure of a primary structural component leads to the collapse of adjoining members and to an overall damage which is disproportionate to the initial cause.

The problem of progressive collapse in civil engineering gained its interest as early as 1968, after the Ronan Point apartment building partial collapse, London, the UK. Right after this event, many research efforts led to developing progressive collapse strategies and methods as well as resulted in first progressive collapse provisions in the UK standards. Even current methods and strategies benefit to a great extent from the approaches developed in that time. As summarised by Moore [54], those provisions helped in many cases to avoid other progressive collapses, afterwards, in the UK. A second and third wave of progressive collapse interest by the civil engineering community appeared after the disproportionate collapse of the Alfred P. Murrah Federal Building (Oklahoma City, 1995) and the total collapse of the World Trade Center towers, both caused by terrorist attacks.

Progressive collapse can be triggered by many different actions. Examples of such actions can be: explosions caused by gas or explosives; impacts of vehicles, ships or planes; earthquakes human errors in the design or construction phase etc. Prediction of such abnormal actions is very difficult and depends on many factors. Designing for example a large frame reinforced concrete building against progressive collapse due to blast loading is a big challenge. Analysing such a building and checking if a progressive collapse could happen or not depends on many assumptions. For example the major unknowns are: how big the explosive charge is (what is the peak pressure), how far from the building the explosive is detonated, and whether the blast affects the corner load-bearing elements of the building or the ones situated in the middle of the building's sides etc.

These difficulties make that there are effectively no provisions in the national codes and standards to design structures to resist external explosions or internal explosions caused by explosives. Thus instead of explicit analysis of a structure to a specific blast load, the current codes, standards and guidelines recommend a threat independent design, that is the design due to an unspecified cause, or to design some elements (key elements) to resist a sufficiently high pressure (e.g. 34 kPa).

Those early developed design approaches for progressive collapse mitigation can be divided into indirect and direct approaches. Indirect approaches consist in applying prescriptive design rules (minimum requirements on strength, continuity, ductility, redundancy), providing resistance to progressive collapse; however progressive collapse behaviour is not addressed explicitly. In other words, indirect design approach addresses the problem by identifying and incorporating into the building system characteristics that enhance robustness, without special consideration of loads or events that could trigger disproportionate collapse. Direct design approaches involve a performance-based approach and consist of the specific local resistance method and the alternate load path method.

This report contains 6 Chapters. Chapter 1 is an introduction to the topic. In Chapter 2 the main terms and definitions related to progressive collapse are presented. Chapter 3 provides a review of procedures and strategies for progressive collapse design based on selected codes and standards, e.g. ASCE 7 [5], BS 5628 [9], BS 5950 [10], BS 6399 [11], BS 8110 [12], EN 1991 [27], and U.S. guidelines, e.g. DoD UFC Guidelines [20], GSA Guidelines [40]. Chapter 4 constitutes a review of research efforts in the field of progressive collapse reported in international journals and conference papers. Different proposals of robustness measures of structures are also presented in this chapter. In Chapter 5 a few examples of progressive collapses of real buildings are described. Finally, Chapter 6 provides a summary and conclusions. Two appendices, containing respectively some useful elements on plastic analysis and on blast loading of structures, are included.

2. Terms and definitions

In different publications, the common used terms regarding progressive collapse can have broader or narrower scope and are sometimes used with slightly different meaning. Therefore, the aim of this chapter is to give a list of the terms with their definitions.

Progressive collapse – the spread of an initial local failure from element to element, resulting eventually, in the collapse of an entire structure or a disproportionately large part of it (ASCE 7 [5]).

Progressive collapse – the spread of local damage from an initiating event, from element to element, resulting in the collapse of an entire structure or a disproportionate large part of it; known as disproportionate collapse (NIST Best Practices [56]).

Progressive collapse – a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause (GSA Guidelines [40]).

Progressive collapse – this term is indirectly defined in the EN 1990 [26], where the code treats the basic requirements a structure should satisfy: "A structure shall be designed and executed in such a way that it will not be damaged by events such as explosions, impact or the consequences of human errors, to an extent disproportionate to the original cause." EN 1990 [26], 2.1(4)

Robustness – the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause (EN 1991-1-7 [28]).

Robustness – the ability of a structure or structural components to resist damage without premature and/or brittle failure due to events like explosions, impacts, fire or consequences of human error, due to its vigorous strength and toughness (GSA Guidelines [40]).

Robustness – insensitivity of a structure to local failure, where "insensitivity" and "local failure" are to be quantified by the design objectives (Starossek and Haberland [69]). Defined in this way, robustness is a property of the structure alone and is independent of possible causes of initial local failure. This definition is in contrast to a broader definition of robustness – as it is given, for instance, in EN

1991-1-7 [28] – which does include possible causes of initial failure. Such a broader definition is close to the term collapse resistance as defined below.

Collapse Resistance – insensitivity of a structure to accidental circumstances, which are low probability events and unforeseeable incidents. The accidental circumstances are to be quantified by the design objectives. Collapse resistance is a property that is influenced by numerous conditions including both structural features and possible causes of initial failure. (Starossek [65], Starossek and Haberland [69]).

Key element – a structural member upon which the stability of the remainder of the structure depends (EN 1991-1-7 [28]).

Key element – structural elements whose notional removal would cause collapse of an unacceptable extent. They should therefore be designed for accidental loads, which are specified in several standards as $34\,\text{kPa}$ (NIST Best Practices [56]).

Localised failure – the part of a structure that is assumed to have collapsed, or been severely disabled, by an accidental event (EN 1991-1-7 [28]).

Continuity – refers to the continuous connection of components as well as the continuous reinforcement of concrete components. Integrity, redundancy and/or local resistance can be improved by continuity. Continuity is thus an element of robustness (Starossek and Haberland [69]).

Damage tolerance – the term is compatible with the term robustness used by Lind [46]. In some other papers, the damage tolerance meaning is narrower and refers to the ability of a structure to resist a continuous local deterioration due to corrosion or similar (Starossek and Haberland [69]).

Ductility – the ability of a component or structural system to withstand large plastic deformations. Ductility has a large influence on progressive collapse and is often listed as a factor which increases the robustness of the structure (Starossek and Haberland [69]).

Ductility – the ability of a structure to remain stable after large deformations (rotations and deflections). There are different means for steel and reinforced concrete structures to provide sufficient ductility. Steel – using steel with high toughness, connections which exceed the strength of the base material. Reinforced concrete structures – confinement of reinforcing steel, continuity through lap splices, maintaining overall structural stability, and creating connections between elements that exceed the strength and toughness of the base members etc. (NIST Best Practices [56]).

Integrity – the term is mainly used in U.S. standards such as ACI 318 [2], ASCE 7 [5], often in relation with prescriptive requirements (like requirements for continuity, ductility, and redundancy). Integrity implies that the structure and its

components remain intact over the intended lifetime of the structure (Starossek and Haberland [69]).

Redundancy – Structural redundancy refers to the multiple availability of load-carrying components or multiple load paths which can bear additional loads in the event of a failure. If one or more components fail, the remaining structure is able to redistribute the loads and thus prevent a failure of the entire system. Redundancy depends on the geometry of the structure and the properties of the individual load-carrying elements (Frangopol and Curley [39]). It is not synonymous with the static indeterminacy. Redundancy is mentioned as an important factor in the design of robust structures and hence the prevention of progressive collapse (EN 1991-1-7 [28]). Redundancy refers particularly to the alternate load path method (Starossek and Haberland [69]).

Redundancy – the incorporation of redundant load paths in the vertical load carrying system to ensure that alternate load paths are available in the event of local failure of structural elements (NIST Best Practices [56]).

Vulnerability – describes the sensitivity of a structure to damage events. A structure is vulnerable if small damages lead to disproportionate consequences. Vulnerability is opposite to robustness and it is a property of the structural system (Starossek and Haberland [69]).

Ties – The loss of a major structural element typically results in load redistributions and member deflections. These processes require the transfer of loads throughout the structure (vertically and horizontally) through load paths. The ability of a structure to re-distribute or transfer loads along these load paths is based in large part on the interconnectivity between adjacent members. This is often called "tying a building together" by using an integrated system of ties in three directions along the principal lines of structural framing. Fig. 3.4 taken from DoD UFC Guidelines [20], illustrates the different types of ties that are typically incorporated to provide structural integrity to a building (NIST Best Practices [56]).

Direct design for progressive collapse – explicit consideration of progressive collapse during the design process through either: alternate load path method or specific local resistance method (ASCE 7 [5]).

Indirect design for progressive collapse – implicit consideration to progressive collapse during the design process through the provision of minimum levels of strength, continuity and ductility (ASCE 7 [5]).

Specific local resistance method (Key element method) – a method that seeks to provide sufficient strength to resist failure from accidents or misuse. In other words, a critical load bearing element is explicitly designed to resist the prescribed load level (ASCE 7 [5]).

Alternate load path method – a method that allows local failure to occur,

but seeks to provide alternate load paths so that the damage is absorbed and major collapse is averted (ASCE 7 [5]).

Risk – a measure of the combination (usually the product) of the probabilities or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence (EN 1991-1-7 [28]).

Consequences – a possible result of an event. Consequences may be expressed verbally or numerically in terms of loss of life, injury, economic loss, environmental damage, disruption to users and the public, etc. Both immediate consequences and those that arise after a certain time has elapsed are to be included (EN 1991-1-7 [28]).

Risk analysis – a systematic approach for describing and/or calculating risk. Risk analysis involves the identification of undesired events, and the causes and consequences of these events (EN 1991-1-7 [28]).

Risk evaluation – a comparison of the results of a risk analysis with the acceptance criteria for risk and other decision criteria (EN 1991-1-7 [28]).

Risk management – systematic measures undertaken by an organisation in order to attain and maintain a level of safety that complies with defined objectives (EN 1991-1-7 [28]).

Risk acceptance criteria – these criteria are normally determined by the authorities to reflect the level of risk considered to be acceptable by people and society. They correspond to acceptable limits to probabilities of certain consequences of an undesired event and are expressed in terms of annual frequencies (EN 1991-1-7 [28]).

Deflagration – propagation of a combustion zone at a velocity that is less than the speed of sound in the unreacted medium (EN 1991-1-7 [28]).

Detonation – propagation of a combustion zone at a velocity that is greater than the speed of sound in the unreacted medium (EN 1991-1-7 [28]).

3. Review of procedures and strategies for progressive collapse design

3.1. British Standards

The United Kingdom was the first country which incorporated the progressive collapse provisions to its standards. The need for this kind of regulations emerged after the Ronan Point partial collapse (see details in Sec. 5.1). General information on how to design structure against progressive collapse is given in BS 6399 [11], while specific provisions for steel, concrete and masonry structures are given in BS 5950 [10], BS 8110 [12] and BS 5628 [9], respectively. Below there are presented main topics of progressive collapse design which can subsequently be used for comparison with other documents.

3.1.1. Load combinations

For bridging design (alternate load path method), British Standards recommend applying the load combinations as follows

$$D + W/3 + L/3 \tag{3.1}$$

where: D - dead load, W - wind load, L - imposed load.

3.1.2. Horizontal ties

Steel structures

Steel elements designed as horizontal ties and their end connections should be capable of resisting factored tensile loads as follows:

- internal ties

$$T_i = 0.5(1.4g_k + 1.6q_k)s_t L \quad \text{but not less than} \quad 75\,\text{kN}, \tag{3.2}$$

where:

g_k – the specified dead load per unit area of the floor or roof,

q_k – the specified imposed floor or roof load per unit area,

L – the span,

s_t – the mean transverse spacing of the ties adjacent to that being checked.

- edge ties

$$T_e = 0.25(1.4g_k + 1.6q_k)s_tL \quad \text{but not less than} \quad 75\,\text{kN}. \tag{3.3}$$

Reinforced concrete structures

- internal ties

$$T_i = \frac{g_k + q_k}{7.5}\frac{l_r}{5}F_t \text{ or } 1.0F_t \tag{3.4}$$

where:

g_k – characteristic dead load (in kN/m^2),

q_k – characteristic imposed floor load,

l_r – greater of the distances between the centres of the columns, frames or walls supporting any two adjacent floor spaces in the direction of the tie under consideration,

F_t – basic strength, lesser of $(20 + 4n_o)$ or 60 kN,

n_o – number of storeys.

- peripheral ties

At each floor and roof level an effectively continuous peripheral tie should be designed, capable to resist a tensile force of $1.0F_t$, located within 1.2 m of the edge of the building or within the perimeter wall.

3.1.3. Vertical ties

Vertical ties should ensure continuous tying of the structure from the lowest to the highest level. The column or a wall designed as a vertical tie should be capable to resist a tensile force equal to the maximum design ultimate dead and imposed load.

3.1.4. Design of bridging elements (alternate load path)

Steel structures

If the conditions for the tie forces cannot be met, the building should be checked to ensure that the notional removal of a column (at each level, one at a time) will not lead to disproportionate collapse.

Reinforced concrete structures

For buildings of 5 or more storeys, when the tie forces criteria cannot be met, the structure should be analysed upon removal of a load-bearing element (a column or a portion of wall between lateral supports).

3.1.5. Key elements

Steel structures

If the conditions for the tie forces are not satisfied and upon a column removal the building is suspected to total collapse or the area of the collapsed portion is greater than 15% or $70\,\mathrm{m^2}$, then that column or element should be designed as a key element. The column or element is deemed as key element if it can resist the pressure of $34\,\mathrm{kN/m^2}$.

Reinforced concrete structures

Similarly to the steel structures, the key element should be capable to withstand a design ultimate load of $34\,\mathrm{kN/m^2}$ from each direction. This design ultimate load value should not include a partial safety factor.

3.2. Eurocodes

The Eurocodes (EN 1990 [26], EN 1991 [27], EN 1992 [29], EN 1993 [30], EN 1994 [31], EN 1995 [32], EN 1996 [33], EN 1997 [34], EN 1998 [35], EN 1999 [36]) are a set of European codes for designing and constructing civil engineering structures. Accidental actions are specifically dealt in EN 1991-1-7 [28], however since the Eurocodes are treated as a whole, there are many references to other parts, in particular, to EN 1990 [26] and EN 1991 [27] etc.

EN 1991-1-7 [28] gives provisions (strategies and rules) for designing buildings against identifiable and unidentifiable accidental actions. However, as it is written in the Eurocode, "EN 1991-1-7 does not specifically deal with accidental actions caused by external explosions, warfare and terrorist activities, or the residual stability of buildings or other civil engineering works damaged by seismic action or fire etc." (EN 1991-1-7 [28], 1.1(6)). Thus when designing a structure against a possible threat of a terrorist attack, the design must be conducted according to provisions for an unspecified accidental action. Some of the material below is included in the rules and other in the informative Annexes.

According to EN 1991-1-7 [28] the strategies for accidental design situations are illustrated in Fig. 3.1. Therefore, if an accidental action is identified we may try to
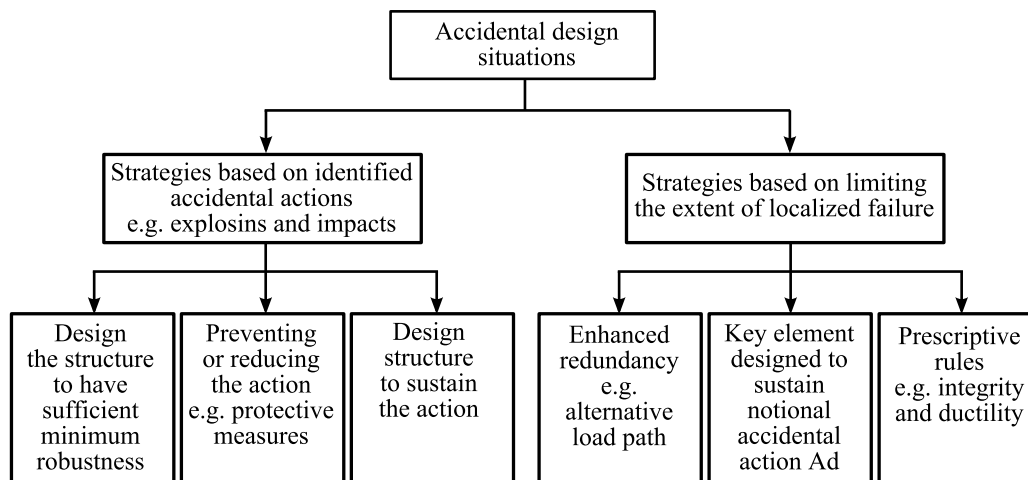
Figure 3.1. Strategies for accidental design situations (EN 1991-1-7 [28])

prevent or reduce the action by protective measures, we can design the structure for sufficient robustness or to sustain the action. On the other hand, if we allow a local damage, then the aim of the design is to either enhance structural redundancy by alternate load path method, or to ensure structural integrity and ductility.

Potential damage can be avoided or limited by appropriate choice of one or more of the following actions:

- avoiding, eliminating or reducing the hazards to which the structure can be subjected,
- selecting a structural form which has low sensitivity to hazards considered,
- selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage,
- avoiding as far as possible structural systems that can collapse without warning,
- tying the structural members together. EN 1990 [26], 2.1(5)

The strategies for accidental design situations depend on three consequences classes defined in EN 1990 [26]. These consequences classes (CC) include:

CC1 – low consequences of failure,
CC2 – medium consequences of failure,
CC3 – high consequences of failure.

EN 1991 [27] assigns accidental design situations for different consequences classes as follows:

CC1 – no particular consideration is necessary for accidental actions other than satisfying rules for stability and robustness given in other Eurocodes (EN 1990 [26] to EN 1999 [36]),

CC2 – depending on specific conditions of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design and detailing rules may be applied,

CC3 – a more detailed consideration of the specific case should be done to determine the level of reliability and the depth of structural analyses required. A risk assessment as well as advanced analysis method (nonlinear dynamic analysis) may be required.

Annex A of the EN 1991-1-7 [28] gives a categorisation of buildings types with regard to consequences classes. A simplified version of the Table A.1 of EN 1991-1-7 [28] can be represented as follows:

CC1 – single occupancy houses not exceeding 4 storeys, agricultural buildings, buildings rarely occupied by people etc.;

CC2a (lower risk group) – 5 storey single occupancy houses, hotels, flats, apartments, other residential buildings, offices not exceeding 4 storeys etc.;

CC2b (upper risk group) – hotels, flats, apartments and other residential buildings greater than 4 storeys but less than 15 storeys etc.;

CC3 – all buildings defined for classes CC2a and CC2b that exceed the limits on area or number of storeys, all buildings occupied by people in significant numbers, stadia for more than 5000 spectators, buildings containing dangerous substances and processes etc.

Based on these categorisation, the following strategies are recommended:

a) for buildings in Consequences Class 1: as mentioned before no specific design procedure is needed other than those for designing and constructing buildings in accordance with the rules in other Eurocodes,

b) for buildings in Consequences Class 2a (lower group): additional procedure include applying appropriate effective horizontal ties, or effective anchorage of suspended walls as defined in 3.2.2,

c) for buildings in Consequences Class 2b (upper group):
   - horizontal and vertical ties as defined in 3.2.2 and 3.2.3 should be provided;
   - the building should be analysed to check if the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall will not cause local damage greater than the specified limits and not cause total collapse. Where the notional removal of such columns and sections of walls would result in exceeding the specified limits for local damage, then those elements should be redesigned or designed as a "key element" (see 3.2.4),

d) for buildings in Consequences Class 3: A systematic risk assessment of the building should be performed, taking into account both foreseeable and unforeseeable hazards according to Annex B of EN 1991-1-7 [28].

In Annex A of the EN 1991-1-7 [28] there are given rules and methods for designing buildings to sustain an extent of localised failure from an unspecified cause without disproportionate collapse.

3.2.1. Load combinations

Accidental actions shall be applied simultaneously in combination with permanent and variable loads in accordance with (EN 1990 [26], 6.4.3.3).

Combination of actions for accidental design situations in the ultimate limit states according to (EN 1990 [26], 6.4.3.3) is as follows

$$\sum_{j\geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1})Q_{k,1} + \sum_{i>1} \psi_{2,i}Q_{k,i} \qquad (3.5)$$

where:
- G – permanent load (dead load),
- P – relevant representative value of a prestressing action (see EN 1992 to EN 1996 and EN 1998 to EN 1999),
- A_d – design accidental action,
- Q – variable load (live load, snow load, wind load),
- ψ_1 – factor for frequent value of a variable action,
- ψ_2 – factor for quasi-permanent value of a variable action.

Accidental action A_d should be taken as an explicit accidental action A_d for fire or impact or can refer to the situation after an accidental event. In this case A_d is equal to zero. Recommended values for ψ_1 and ψ_2, depending on the building categories, can be found in Table A1.1 of Annex A EN 1990 [26].

In the paragraph EN 1990 [26], 4.1.2(8), it reads as follows "For accidental actions the design value A_d should be specified for individual projects based on EN 1991 [27]".

When analysing a structure in a quasi-static way, the dynamic effects can be included by applying an equivalent dynamic amplification factor to the static actions, EN 1990 [26], 5.1.3(3). However it is not specified in the Eurocodes what value for the dynamic amplification factor is recommended in the case of accidental actions.

The accidental actions to be considered depend on:

- the measures taken to prevent or reduce the severity of an accidental action,
- the probability of occurrence of the identified accidental action,
- the consequences of failure due to the identified accidental action,
- public perception,
- the level of acceptable risk.

A localised failure due to accidental actions may be acceptable, provided it will not endanger the stability of the whole structure, and that the overall load bearing capacity of the structure is maintained and allows necessary emergency measures to be taken.

Measures should be taken to reduce the risk of accidental actions and these measures should include one of more of the following strategies: preventing the

action from occurring, protecting the structure against the effects of an accidental action by reducing the effects of the action on the structure, ensuring that the structure has sufficient robustness.

3.2.2. Horizontal ties

For framed structures, continuous internal ties, including their end connections, should be capable of resisting a design tensile load of a value

$$T_i = 0.8(g_k + \psi q_k)sL \quad \text{or} \quad 75\,\text{kN} \quad \text{whatever is greater} \tag{3.6}$$

similarly for perimeter ties a design tensile force is given as

$$T_p = 0.4(g_k + \psi q_k)sL \quad \text{or} \quad 75\,\text{kN} \quad \text{whatever is greater} \tag{3.7}$$

where:
s – the spacing of ties,
L – the span of the tie,
ψ – the relevant factor in the expression for combination of action effects for the accidental design situation (as in Eq. (3.5)).

For load-bearing walls, how the ties should be incorporated into the building depends on the consequences class. For CC 2 buildings (Lower Risk Group), adequate robustness is provided by adopting a cellular form of construction designed to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls. For CC 2 buildings (Upper Risk Group), continuous horizontal ties should be provided in the floors. These should be internal ties distributed throughout the floors in both orthogonal directions and peripheral ties extending around the perimeter of the floor slabs within 1.2 m width of the slab. The design tensile forces in the ties should be calculated as follows:

- for internal ties

$$T_i = \frac{F_t(g_k + \psi q_k)}{7.5} \cdot \frac{z}{5} \quad \text{or} \quad T_i = F_t \quad \text{whatever is greater,} \tag{3.8}$$

- for peripheral ties

$$T_p = F_t, \tag{3.9}$$

where:
F_t – 60 kN/m or $20 + 4n_s$ [kN/m], whichever is less,
n_s – number of storeys,
z – smaller value of: $5 \cdot H$ or the greatest distance [m] in the direction of the tie, between the centers of the columns or other vertical load-bearing members whether the distance is spanned by a single slab or by a system of beams and slabs,
H – clear storey height.

3.2.3. Vertical ties

All vertical ties (for frame and wall structures) should be continuous from the foundations to the roof level.

For frame structures, vertical ties should be capable to resist an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. It should be noted that this accidental design loading should not act simultaneously with permanent and variable actions that may be acting on the structure.

For wall structures, vertical ties may be deemed effective if:

a) for masonry walls their thickness is at least 150 mm and if they have minimum compressive strength of 5 N/mm^2 (see EN 1996 [33]),
b) the clear height of the wall, measured in meters between faces of floors or roof does not exceed $20t$, where t is the thickness of the wall in meters,
c) if vertical ties resist the following force

$$T = \frac{34A}{8000}\left(\frac{H}{t}\right)^2 \quad \text{or} \quad 100\,\text{kN/m} \quad \text{of wall, whichever is greater,} \qquad (3.10)$$

   where A – the cross-sectional area in mm^2 of the wall measured on plan, excluding the non load bearing leaf of a cavity wall;
d) the vertical ties are grouped at 5 m maximum centres along the wall and occur no greater than 2.5 m from an unrestrained end of the wall.

3.2.4. Key elements

For building structures, a key element should resist an accidental design action of A_d applied in horizontal and vertical directions (one direction at a time). Such accidental design loading should be applied in accordance with expression (6.11b of the EN 1990 [26], here see Eq. (3.5)) and may be concentrated or distributed load. The recommended value of A_d for building structures is 34 kN/m^2.

3.2.5. Risk assessment

For category CC3 of buildings, Eurocode EN 1991-1-7 [28] requires a risk assessment for a building. Risk is defined as a measure of the combination of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence and is expressed as

$$R = \sum_{i=1}^{N_H} p(H_i) \sum_{j}^{N_D} \sum_{k=1}^{N_S} p(D_j|H_i) \cdot p(S_k|D_j) \cdot C(S_k), \qquad (3.11)$$

where N_H – number of different hazards, N_D – number of ways the hazards may damage the structure, N_S – number of adverse states (S_k) into which the damage structure can be discretised, $C(S_k)$ – consequences of an adverse state, $P(H_i)$ – probability of occurrence (within a reference time interval) of the i-th hazard, $p(D_j|H_i)$ – the conditional probability of the j-th damage state of the structure given the i-th hazard and $p(S_k|D_j)$ – the conditional probability of the k-th adverse overall structural performance S given j-th damage state.

Analysing Eq. (3.11) there are the following possible strategies to control and mitigate the risk:

- reduction of the probability that a hazard occur (reduction of $P(H)$). For example if there is a risk that a ship impacts a bridge pier, creation of artificial islands around the bridge pier will mitigate the risk.
- reduction of the probability of significant damages for given hazards (reduction of $P(D|H)$).
- reduction of the probability of adverse structural performance given structural damage (reduction of $P(S|H)$).

Thus, risk analysis of structures subject to accidental actions involves the following steps:

1. Assessment of the probability of occurrence of different hazards with their intensities.
2. Assessment of the probability of different states of damage and corresponding consequences for given hazards.
3. Assessment of the probability of inadequate performance of the damaged structure together with corresponding consequences.

3.2.6. Dynamic design against impact

Annex C of the EN 1991-1-7 [28] gives guidance for the approximate dynamic design of structures to accidental impact by road vehicles, rail vehicles and ships based on simplified or empirical models.

First, the general impact dynamics theory is considered, where impacts are idealised and grouped into two types, namely hard impacts (the energy is dissipated by the impacting object) and soft impacts (the structure absorbs the impact energy by structure's deformation). For hard impacts, EN 1991-1-7 formulates an expression for the maximum resulting dynamic interaction force (F) in function of the object velocity at impact (v_r), the equivalent elastic stiffness of the object (k) and the mass of the impacting object (m). This maximum dynamic interaction force is for the outer surface of the structure, while for the structure itself the dynamic effects can be greater and should be included by applying the dynamic amplification factor. For soft impacts, the same formula for the maximum dynamic interaction force can be used, however for k, the stiffness of the structure should be taken. There is also

formulated a provision that the structure should have sufficient ductility to be able to absorb the total kinetic energy by plastic deformation.

The second part of the Annex C of the EN 1991-1-7 is devoted to specific provisions for impacts by road vehicles and ships giving formulas or values for the velocities of impact (v_r) and approximate design values for the dynamic interaction forces (F_d) depending on different factors such as: where the vehicles travel, the mass of the vehicles, distance of vehicles from the road lanes, size and mass of ships, whether the ships travel on inland or sea waterways etc.

3.2.7. Internal explosions

Annex D of the EN 1991-1-7 [28] provides guidance on how to deal with:

- dust explosions in rooms, vessels and bunkers,
- natural gas explosions,
- explosions in road and rail tunnels.

For dust explosions in rooms, vessels and bunkers, the EN 1991-1-7 gives:

- material parameters K_{St} (which characterise the confined explosion behaviour) for most common types of dust and,
- a formula for the venting area of cubic, elongated rooms, vessels and bunkers, as well as for rectangular enclosures.

For natural gas explosions, EN 1991-1-7 gives formulae for a nominal equivalent static pressure as the loading a structure should withstand.

For explosions in road and rail tunnels, EN 1991-1-7 provides expressions for the pressure time function in the cases of detonation and deflagration (see Chapter 2).

3.3. ASCE 7-05

The American Society of Civil Engineers, ASCE 7 [5] discusses general design specifications for reducing the potential of progressive collapse, however, no specific requirements are given. Similarly no U.S. building codes provide specific design requirements with regard to progressive collapse.

The commentary to the ASCE 7 [5] provides detailed discussion on general structural integrity. It gives a list of possible methods for progressive collapse design such as direct and indirect design approaches. Direct design approaches includes alternate load path method and specific local resistance method, whereas indirect design approach is based on implicit consideration of progressive collapse resistance by ensuring minimum levels of strength, continuity and ductility. Similarly to the Eu-

rocodes and British Standards, there is no provision on what dynamic amplification factor should be used when equivalent static methods are used.

3.3.1. Load combinations

ASCE 7 [5] specifies the following load combinations:

- for specific local resistance method

$$1.2D + A_k + (0.5L \text{ or } 0.2S) \quad \text{or} \tag{3.12}$$
$$(0.9 \text{ or } 1.2)D + A_k + 0.2W_n, \tag{3.13}$$

- for alternate load path method (checking if residual load-carrying capacity upon notional removal of a selected load-bearing element)

$$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W_n, \tag{3.14}$$

where:
- D – dead load,
- L – live load,
- W_n – wind load,
- S – snow load,
- A_k – load effect resulting from an extraordinary event to be specified by the authority having jurisdiction.

3.4. GSA Guidelines

The US General Services Administration (GSA) Guidelines [40] do not mention anything about designing key elements or designing based on tie forces.

GSA Guidelines permit to perform both linear and nonlinear analysis techniques, however the latter is regarded as much more difficult and can only be performed by experienced structural analyst with advanced structural engineering knowledge. There is only one paragraph devoted to nonlinear procedure with general remarks and no detailed guidance. However acceptance criteria for nonlinear analysis are given.

GSA Guidelines provide minimum defended stand-off distances for different types of construction (reinforced concrete, steel, masonry, precast, wood) depending on required level of protections.

GSA Guidelines [40] use a flow-chart methodology to determine if a designed building should have additional resistance for progressive collapse or can be exempt from the process of progressive collapse design. The exemption is based on certain criteria such as building occupancy, building category (reinforced concrete building,

steel frame building etc.), number of storeys, seismic zone, local structural features and global structural features. Whether positive or negative the answer is, the building is exempt or not, from further consideration of the potential for progressive collapse. The purpose of the detailed analysis is to reduce the probability of progressive collapse for new construction and identify the potential for progressive collapse in existing construction.

GSA Guidelines state that low and medium rise buildings can be designed according to the simplified analysis procedure (linear procedure) while other building (more than 10 storeys and with atypical structural configuration) should use a nonlinear procedure.

GSA Guidelines give only general considerations for nonlinear procedure analysis with acceptance criteria for reinforced concrete and steel structures while two separate chapters are devoted to linear procedures of reinforced concrete and steel structures.

There are also presented detailed information on design and analysis of reinforced concrete and steel structures. This includes guidance on how to design, model and analyse the structures.

3.4.1. Design guidance

For reinforced concrete structures special attention should be paid to structural redundancy, detailing to provide structural continuity and ductility, capacity for resisting load reversals and capacity for preventing shear failure.

Structural redundancy using redundant vertical and lateral force resisting systems enables developing alternate load paths and forming multiple plastic hinges which can help to prevent total collapse.

When a vertical load-bearing member is missing it is important that main structural elements are capable to carry two spans. This implies good beam-to-beam continuity across removed element and the ability of structural elements to develop large deformations.

The capacity for resisting load reversals should be achieved in such a way that structural members (girders, beams) have additional reinforcement in the zones of compressed fibres since, for instance, after removing a column, the previous negative moment above the column transforms into a positive moment.

For steel structures, the guidelines emphasise such aspects as beam-to-beam continuity, connection resilience, connection redundancy, connection rotational capacity.

3.4.2. Analysis techniques

The analysis techniques presented in GSA Guidelines should mainly use linear elastic, static analysis preferably on a 3 dimensional models and consist of:

- removing a vertical load-bearing member instantaneously and
- applying load combinations multiplied by the factor of 2 accounting for the dynamic and nonlinear effects (see 3.4.3 for more details).

3.4.3. Load combinations

The structure should be analysed using the following load combinations applied to the whole structure together with an instantaneous loss of primary vertical support

$$2(D + 0.25L) \quad \text{static analysis,} \tag{3.15}$$
$$D + 0.25L \quad \text{dynamic analysis,} \tag{3.16}$$

where: D - dead load, L - live load.

For frame structures a column removal should be analysed for one floor above ground. The exterior locations of columns to be removed include:

- the middle of the short side of the building,
- the middle of the long side of the building,
- the corner of the building.

If there is an underground parking area or uncontrolled public ground floor, the column to be removed should be interior to the perimeter column lines.

For wall structures the considered part of removed load-bearing wall include one structural bay or 30 ft of an exterior wall section (whichever is less) located at:

- the middle of the short side of the building,
- the middle of the long side of the building,
- the corner of the building.

Again if there is an underground parking area or uncontrolled public ground floor, the instantaneously removed section should be one bay or 30ft of an interior wall section (whichever is less) close to the perimeter of the bearing wall line.

The removal of the vertical element for dynamic analysis should be instantaneous and in any case the removal time should not exceed 1/10 of the period corresponding to the structural response mode for the vertical element removal.

3.4.4. Analysis criteria

The maximum allowable size of damage caused by the instantaneous removal of a primary *exterior* vertical member shall be limited to the structural bays directly associated with the instantaneously removed or $1800\,\text{ft}^2$ ($167\,\text{m}^2$) at the floor level directly above the instantaneously removed vertical member (whichever is the smaller area).

In the case of the instantaneous removal of a primary *interior* vertical member the corresponding allowable extend of damage should be limited to the structural bays directly associated with the instantaneously removed or $3600\,\text{ft}^2$ ($334\,\text{m}^2$) at the floor level directly above the instantaneously removed vertical member (whichever is the smaller area).

The same above requirements are applicable for reinforced concrete and steel structures.

3.4.5. Acceptance criteria

Satisfying acceptance criteria for linear static analysis consist in obtaining the actual internal forces caused by load combinations in an analysed structure and compare them with member capacities. To this end, an indicator DCR (Demand-Capacity Ratio) is defined by

$$\text{DCR} = \frac{Q_{UD}}{Q_{CE}} \tag{3.17}$$

where:
Q_{CE} – expected ultimate, unfactored capacity,
Q_{UD} – acting force (demand) in structural member or joint (bending moment, shear or axial force).

For reinforced concrete structures allowable DCR values are as follows: DCR $\leq$ 2.0 for typical structural configurations and DCR $\leq$ 1.5 for atypical structural configurations. For steel structures allowable DCR values depend on section compactness[1] and are in the range (1.0–3.0) for typical structural configurations. The criteria for atypical structural configurations are multiplied by 0.75, but not less than 1.0.

Acceptance criteria for nonlinear analysis in terms of ductility and rotation limits are defined in table 2.1 of GSA Guidelines. The table gives values in terms of rotations and ductility for reinforced concrete, steel, unreinforced and reinforced masonry as well as for wood structures. Fig. 3.2 and 3.3 illustrate how to define rotations in beams and frames.

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[1] flange compactness $b_f/(2t_f)$, web compactness h/t_w, where: b_f is the width of the compressed flange, t_f is the thickness of the compressed flange, h is the height of the section and t_w is the thickness of the web.
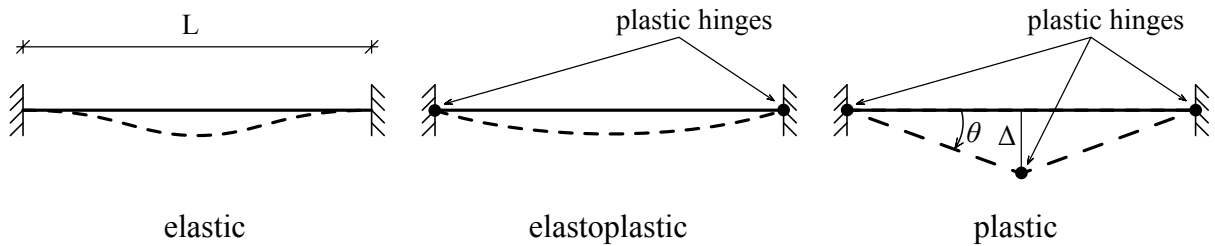
Figure 3.2. Measurement of θ after formation of plastic hinges (from GSA Guidelines [40])
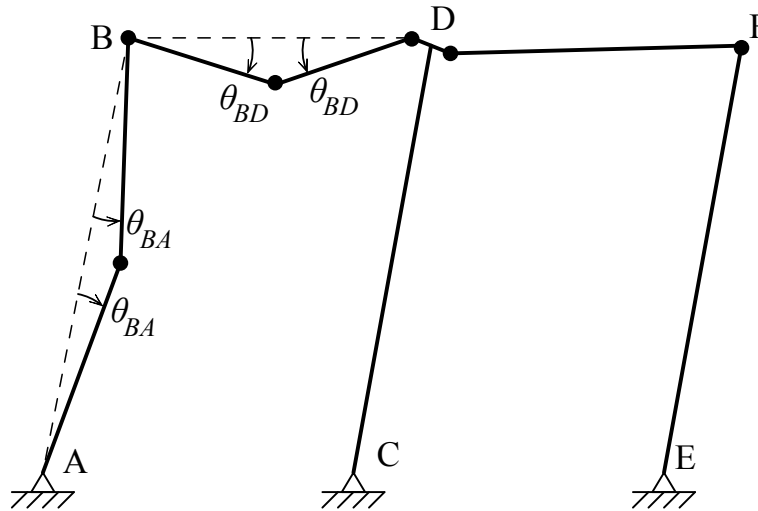
Figure 3.3. Sideways and member end rotations (θ) for frames (from GSA Guidelines [40])

3.5. UFC 4-023-03

The US Department of Defence, Unified Facilities Criteria, DoD UFC Guidelines [20] give the design requirements to mitigate the potential of progressive collapse for new and existing DoD facilities and this guidelines follow other documents of DoD such as DoD UFC 4-010-01 [18], DoD UFC 4-010-02 [19].

DoD UFC Guidelines [20] provide progressive collapse design procedures of two different levels. The first level of progressive collapse uses the provision of tie forces which are based on a catenary response of the structure, while the second level refers to the alternate load path method, in which the building must bridge over a removed element.

However unlike many other documents, DoD UFC Guidelines [20] do not say anything about key elements. The guidelines say that even though other design method for identified and specific threat is used, the progressive collapse requirements of these guidelines still must be met.

The applied level of progressive collapse design is related to the level of protection which must be delivered to the designer by the project planning team. DoD UFC Guidelines specify fours level of protection and assigns appropriate progressive collapse design procedures. For Very Low Level of Protection (VLLOP) and Low Level

of Protection (LLOP) only indirect design is required, by satisfying given levels of tie forces. However, when this condition cannot be satisfied, then the alternate load path method must be used. For Medium Level of Protection (MLOP) and High Level of Protection (HLOP), the alternate load path method should be applied in addition to the tie force method. Moreover for MLOP and HLOP additional ductility requirements should be met for ground floor perimeter vertical load-bearing elements.

According to DoD UFC Guidelines the majority of new and existing DoD structures will fall into the group of VLLOP and LLOP. Thus only tie force method will be used, whose criteria in the majority of the cases will be met without difficulty.

3.5.1. Load combinations

DoD UFC Guidelines [20] define the load combinations indicated below. For static analyses, to account for dynamic effects regarding the removal of a load-bearing element, the dynamic amplification factor is equal to 2 and the appropriate load combinations applied only to the elements of bays related to the removed element and for all storeys of that bays are as follows

$$2(1.2D + 0.5L + 0.2W) \tag{3.18}$$
$$2(1.2D + 0.2S + 0.2W) \tag{3.19}$$

For other structural elements in the static analyses, the load combinations are

$$1.2D + 0.5L + 0.2W \tag{3.20}$$
$$1.2D + 0.2S + 0.2W \tag{3.21}$$

where:
  D – dead load,
  L – live load,
  W – wind load,
  S – snow load.

For dynamic analyses, the load combinations presented above in Eq. (3.20) or (3.21) should be used.

3.5.2. Linear Static Analysis Procedure

In a Linear Static Analysis, the following steps are performed. it should be noted that a second order or P-Δ analysis is required.

1. For alternate path analyses for load-bearing elements that do not have adequate vertical tie force capacity, remove the element from the structural model in accordance with the material-specific requirements. For alternate path analyses of MLOP and HLOP structures, remove the column or load-bearing wall.

2. Apply the loads.

3. After the analysis is performed, compare the predicted element and connection forces and deformations against the acceptability criteria that are shown generically in Table 3-1 of UFC 4-023-03. To demonstrate compliance with the acceptability criteria, a software package with modules that perform building code checks may be used, providing the modules can be tailored to check the criteria in Table 3-1. Confirm that all material-specific code provisions for bracing, compactness, flexural-axial interaction, etc., are met.

4. If none of the structural elements or connections violates the acceptability criteria, the analysis is complete and satisfactory resistance to progressive collapse has been demonstrated. If any of the structural elements or connections violate the acceptability criteria, perform the following procedure:

   a) Modify the geometry or material properties of the model, (i.e., remove elements and/or insert hinges and constant moments).

   b) If an element was shown to fail, redistribute the element's loads.

   c) Re-analyse this modified model and applied loading, starting from the unloaded/undeformed condition.

   d) At the end of the re-analysis, assess the resulting damaged state and compare with the damage limits. If the damage limits are violated, re-design and re-analyse the structure, starting with Step 1. If the damage limits are not violated, compare the resulting internal forces and deformation of each element and connection with the acceptability criteria

   e) If any of the acceptability criteria are violated in the new analysis, repeat this process (steps (a) through (e)), until the damage limits are violated or there are no more violations of the acceptability criteria. If the damage limits are violated, re-design and reanalyse the structure, starting with Step 1. If the damage limits are not violated and no new elements failed the acceptability criteria, then the design is adequate

3.5.3. Nonlinear Static Analysis Procedure

In a Nonlinear Static Analysis, the following steps are performed.

1. For alternate path analyses for load-bearing elements that do not have adequate vertical tie force capacity, remove the element from the structural model in accordance with the material-specific requirements. For alternate path analyses of MLOP and HLOP structures, remove the column or load- bearing wall.

2. Apply the loads using a load history that starts at zero and is increased to the final values. Apply at least 10 load steps to reach the total load. The software must be capable of incrementally increasing the load and iteratively reaching convergence before proceeding to the next load increment.

3. As the analysis is performed, compare the predicted element and connection forces and deformations against the acceptability criteria that are shown generically in Table 3-1 of UFC 4-023-03. To demonstrate compliance with the acceptability criteria, a software package with modules that perform building code checks may be used, providing the modules can be tailored to check the crite-

ria in Table 3-1. Confirm that all material-specific code provisions for bracing, compactness, flexural-axial interaction, etc., are met.

4. If none of the structural elements or connections violates the acceptability criteria during the loading process, the analysis is complete and satisfactory resistance to progressive collapse has been demonstrated. If any of the structural elements or connections violate the acceptability criteria, perform the following procedure:

   a) At the point in the load history when the element or connection fails the acceptability criteria, remove the element or connection.

   b) If an element was shown to fail, redistribute the element's loads.

   c) Restart the analysis from the point in the load history at which the element or connection failed and the model was modified. Increase the load until the maximum load is reached or until another element or connection violates the acceptability criteria.

   d) At each point at which the analysis is halted, check the predicted damage state against the damage limits. If the damage limits are violated, re-design and re-analyse the structure, starting with Step 1.

   e) If the damage limits are not violated and the total load has been applied, the design is adequate. If the damage limits are not violated but one of the acceptability criteria was violated in the re-started analysis, repeat this process (Steps (a) through (e)), until the total load is applied or the damage limits are violated.


3.5.4. Nonlinear Dynamic Analysis Procedure

In a Nonlinear Dynamic Analysis, the following steps are performed:

1. Distribute the mass of the structure throughout the model in a realistic manner; lumped masses are not allowed, unless to represent mechanical equipment, pumps, architectural features, and similar items. Distribute mass along beams and column as mass per unit length; for slabs and floors, represent the mass as mass per unit area. If any portion of the structure is represented by solid elements, distribute the mass as mass per unit volume.

2. Prior to the removal of the load-bearing element, bring the model to static equilibrium under the loads; the process for reaching equilibrium under gravity loads will vary with analysis technique.

3. With the model stabilised, remove the appropriate load-bearing element instantaneously. For alternate path analyses for load-bearing elements that do not have adequate vertical tie force capacity, remove the element in accordance with the material-specific requirements. For alternate path analyses of MLOP and HLOP structures, remove the column or load-bearing wall.

4. Continue the dynamic analysis until the structure reaches a steady and stable condition (i.e., the displacement history of the model reaches a near constant value, with very small oscillations and all material and geometric nonlinear processes have halted).

5. During or after the analysis, compare the predicted element and connection forces and deformations against the acceptability criteria that are shown generically in

Table 3-1 of UFC 4-023-03. To demonstrate compliance with the acceptability criteria, a software package with modules that perform building code checks may be used, providing the modules can be tailored to check the criteria in Table 3-1. Confirm that all material-specific code provisions for bracing, compactness, flexural-axial interaction, etc, are met.

6. If none of the structural elements or connections violates the acceptability criteria during the dynamic motion of the structure, the analysis is complete and satisfactory resistance to progressive collapse has been demonstrated. If any of the structural elements or connections violate the acceptability criteria, perform the following procedure:

   a) At the point in the load history when the element or connection fails the acceptability criteria, instantaneously remove the element or connection from the model.

   b) If an element was shown to fail, redistribute the element's loads.

   c) Restart the analysis from the point in the load history at which the element or connection failed and the model was modified. Continue the analysis until the structural model stabilises or until another element or connection violates the acceptability criteria.

   d) For each time at which the analysis is halted due to violation of an element acceptability criteria, check the damage limits. If the damage limits are violated, stop the analysis and re-design and re-analyse the structure, starting with Step 1.

   e) If the damage limits are not violated and the structural model stabilises, the design is adequate. If the damage limits are not violated but one of the acceptability criteria was violated in the re-started analysis, repeat this process (Steps A through E) until the structure reaches a stable condition or the damage limits are violated.
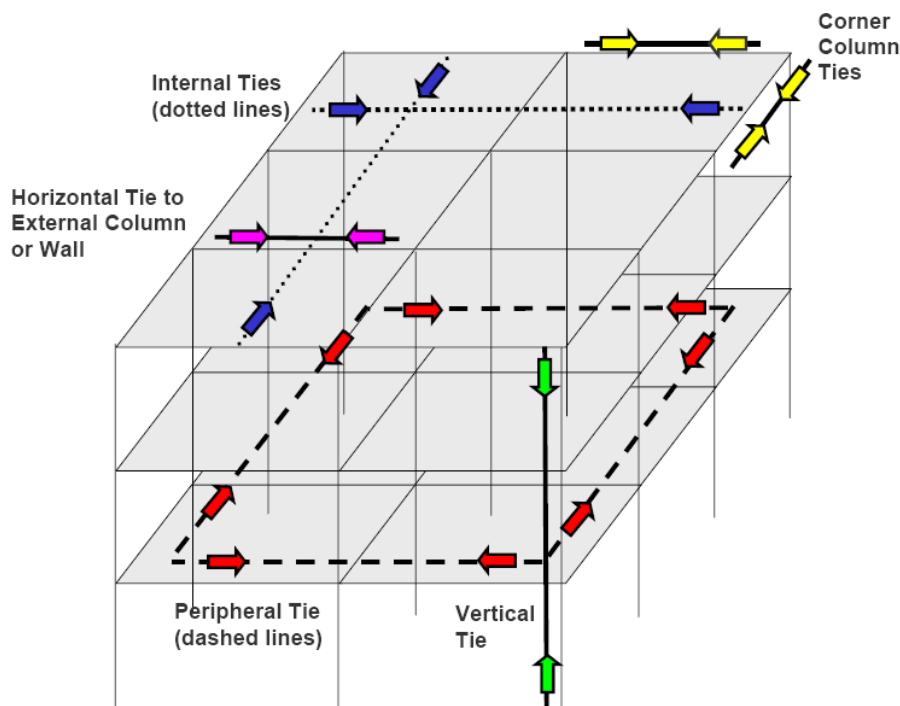
Figure 3.4. Tie forces in a frame structure (UFC 4-023-03)

3.5.5. Internal ties

Specific formulas for internal tie forces are given for different types of structures as follows:

a) reinforced concrete

$$T_i = \frac{1.0D + 1.0L}{7.5}\frac{l_r}{5}F_t \quad \text{or} \quad T_i = 1.0F_t \tag{3.22}$$

where:

D – dead load,

L – live load,

l_r – greater of the distances between the centres of the columns, frames or walls supporting any two adjacent floor spaces in the direction of the tie under consideration,

F_t – basic strength, lesser of $(20 + 4n_o)$ or $60\,\text{kN}$,

n_o – number of storeys

b) steel structures

$$T_i = 0.5(1.2D + 1.6L)s_t L_l \quad \text{but not less than} \quad 75\,\text{kN} \tag{3.23}$$

L_l - span, s_t - mean transverse spacing of the ties adjacent to the ties being checked

c) masonry structures

$$T_i = \frac{1.0D + 1.0L}{7.5}\frac{L_a}{5}F_t \quad \text{or} \quad T_i = 1.0F_t \tag{3.24}$$

L_a - lesser of the greatest distance in the direction of the tie between the centres of columns or the other vertical load-bearing members where this distance is spanned by a single slab or by a system of beams and slabs or $5h$ (h - clear storey height) F_t - basic strength, lesser of $20 + 4n_o$ or 60 n_o - number of storeys including ground and basement

d) wood structures

$$T_i = \frac{1.0D + 1.0L}{3.1}\frac{l_r}{4.6}F_t \quad \text{or} \quad T_i = 1.0F_t \tag{3.25}$$

l_r - greater of the distances between the centres of the columns, frames or walls supporting any two adjacent floor spaces in the direction of the tie under consideration, F_t - basic strength, lesser of $7.3 + 1.46n_o$ or 21.9 n_o - number of storeys

e) cold-formed steel structures

$$T_i = \frac{1.0D + 1.0L}{3.1}\frac{l_r}{4.6}F_t \quad \text{or} \quad T_i = 1.0F_t \tag{3.26}$$

l_r - greater of the distances between the centres of the columns, frames or walls supporting any two adjacent floor spaces in the direction of the tie under consideration, F_t - basic strength, lesser of $7.3 + 1.46n_o$ or 21.9 n_o - number of storeys

$$F_t = \min : (20 + 4n_0) \quad \text{or} \quad 60\,\text{kN} \tag{3.27}$$

3.5.6. Perimeter ties

Similarly, regulations for perimeter ties are defined for the following types of structures:

a) reinforced concrete structures
   At each floor and roof level, ensure a continuous tie of a design strength equal to $1.0F_t$ located within $1.2\,\mathrm{m}$ of building edges or within the perimeter wall.
   Each external column and, if the peripheral tie is not located within the wall, every meter length of external wall carrying vertical load must be anchored or tied horizontally into the structure at each floor or roof level with a required tensile strength equal to the greater of
   - the lesser of $2.0F_t$ or $(l_s/2.5)F_t$
   - 3% of the largest factored vertical load, carried by the column or wall at that level, due to conventional design load combinations
   where l_s is the floor to floor height.
b) steel structures

$$T_p = 0.25(1.2D + 1.6L)s_tL_l \quad \text{but not less than} \quad 75\,\mathrm{kN} \qquad (3.28)$$

All columns must be continuous through each beam-to-column connection. All column splices must provide a design tie strength equal to the largest factored vertical dead and live load reaction (from all load combinations used in the design) applied to the column at any single floor level located between that column splice and the next column splice down or the base of the column.

3.5.7. Modelling of plastic hinges

For linear static analysis, if the calculated moment is greater than the nominal moment strength and it is verified that the element is capable of forming a plastic hinge, insert an equivalent plastic hinge into the model by inserting a discrete hinge in the member at an offset from the member end and add two constant moments, one at each side of the new hinge, in the appropriate direction for the acting moment. The magnitude of the constant moments is equal to the determined plastic moment capacity of the element. For the determination of the plastic hinge the guidelines recommend engineering analysis and judgement or the guidance provided for seismic connections in FEMA [38]. For nonlinear static and dynamic analysis, use software capable of representing post-peak flexural behaviour and considering interaction effects of axial loads and moment. Ensure that shear failure will not occur before developing the full flexural design strength.

3.5.8. Updates from 2009 and 2010

DoD UFC Guidelines [21] has been updated two times, however the second change is minor. The first update of the document resulted from the new test

data and analytical model for steel beam-to-column connections, wood structure under blast damage and collapse loading, reinforced concrete slab response to large deformations, as well as load and dynamic increase factors to account for inertia force, nonlinear geometry and material behaviour. The other reasons for the update concerned contradictions and ambiguities in terminology for structural concept and guidance for linear static, nonlinear static, linear dynamic and nonlinear dynamic methods. In particular, the following changes have been made:

- the levels of protection have been replaced by occupancy categories
- tie force method have been revised (including force values and locations of tie forces),
- in Occupancy Category II, the alternate load path method can be used instead of tie force method,
- modeling parameters and acceptance criteria have been adopted from ASCE 41 Seismic Rehabilitation of Existing Buildings,
- "m-factor" approach for Linear Static analysis has been implemented,
- load increase factors for linear static models and dynamic increase factors for nonlinear static models have been included,
- the additional ductility requirements have been replaced with enhanced local resistance
- the three example problems (reinforced concrete, steel, and wood) have been revised according to the updated UFC 4-023-03.

4. Research papers on progressive collapse

There are already a few literature reviews concerning the problem of progressive collapse, however each of them, being limited in the number of pages, summarises only some aspects of progressive collapse. For example, Mohamed [53] and Nair [55] provide a summary of several research papers on progressive collapse, give a short comparison of codes and standards and present the well known examples of progressive collapse.

4.1. Probability of progressive collapse

Ellingwood and Dusenberry [25] and Ellingwood [24] introduced a formula to assess the probability of progressive collapse as follows

$$P(C) = P(C|DH) \cdot P(D|H) \cdot P(H) \tag{4.1}$$

where:

$P(C)$	– the probability of progressive collapse,
$P(H)$	– the probability of the occurrence of a hazard H,
$P(D|H)$	– the probability of local damage D as a result of a hazard H,
$P(C|DH)$	– the probability of progressive collapse C of the structure as a result of local damage D caused by hazard H.

Starossek and Haberland [69] gave a good illustration of this formula together with assigned appropriate terms (see Fig. 4.1). Considering the above Eq. (4.1) and Fig. 4.1, the probability of progressive collapse can be minimised in three ways, namely by: controlling abnormal events, controlling local element behaviour and/or controlling global system behaviour. Controlling abnormal events by structural engineers is normally very difficult. However engineers can influence the local and global system behaviour e.i. $P(D|H)$ and $P(C|DH)$.

Figure 4.1. Terms in the context of progressive collapse (from [69])

4.2. Inadequacy of current design methods for progressive collapse resistance

According to Starossek [65], there is an inadequacy of current design methods for progressive collapse resistance which can be summarised as follows:

- Current design codes are based on the consideration of local instead of global failure. Global structural safety against the collapse of the entire system or a major part of it is a function of the safety of all the elements against local failure. Various types of structures can respond differently to local failure. Referring to Eq. (4.1), the part $P(C|DH)$ is not considered in the procedures of current design standards.
- The second shortcoming of current design methods is that low-probability events and unforeseeable incidents - i.e., events E for which P(E) is very small - are not taken into account. Starossek argues that for a slender high-rise building, an initial local failure is the simultaneous failure of all vertical load-bearing elements of a storey, thus the probability of collapse is the sum of failure probability of all elements. And if the number of storeys is large enough, even very low probabilities of local failure resulting from accidental circumstances can sum up to a probability of global failure large enough to be seriously considered.
- The third inadequacy of current design procedures lies in the fact that the probabilistic concept requires the specification of acceptable failure probabilities. So far the target failure probabilities of probabilistic design codes have been derived from previous deterministic design codes. Taking into account that a potential progressive collapse can entail huge losses, it would be difficult to reach consensus from the society on acceptable value for the probability of progressive collapse. It seems that this problem can be omitted by not undergoing this question to the public opinion.

## 4.3. Examples of applying different strategies of progressive collapse design

Starossek and Wolff [70] give a concise overview of direct design strategies for progressive collapse using a simplified model of the Alfred P. Murrah Federal Building (see Fig. 4.2). In the specific local resistance method a local damage is not allowed so critical load-bearing elements must be designed to resist a prescribed level of loading. This can be illustrated in Fig. 4.3 where columns of the lowest storey are designed to resist specific accidental action (blast loading, car collision etc.). Other ways to prevent local failure are to provide minimum stand-off distance (see for example DoD UFC 4-010-02 [19]) by special barriers preventing load-bearing elements of a structure from vehicle impacts or control of public access (see Fig. 4.4).

On the other hand, in the alternate load path method, some local damage is allowed but then the structure must be designed in such a way that a new load path
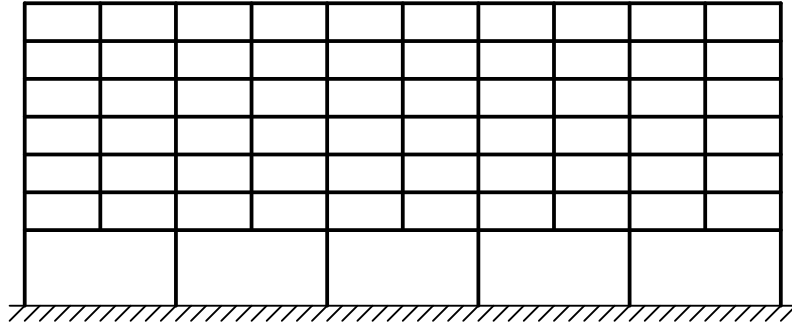
Figure 4.2. Initial frame structure
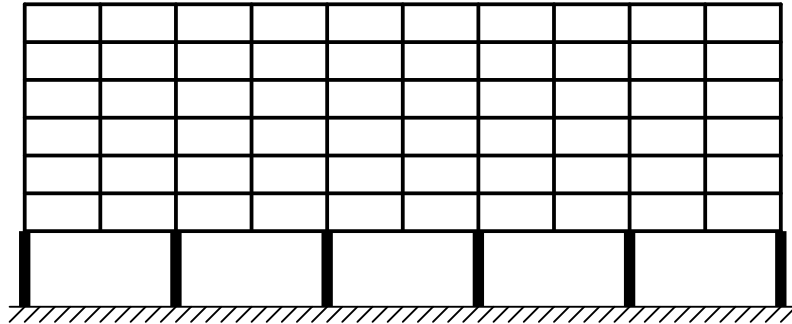
Figure 4.3. Specific local resistance

could be developed to bridge over the missing load-bearing member(s) (see Fig. 4.5). Using one of the alternate load path methods can result in either modification of the initial structural system, for example, by designing more load-bearing elements (columns) as illustrated in Fig. 4.6 or strengthening the transfer girders as shown in Fig. 4.7

In another paper, Starossek [67] considers complex progressive collapse strategies which can be applied to tall buildings. These strategies include: nonstructural protective measures, specific local resistance, alternative paths, isolation of collapsing sections and prescriptive design rules. For specific local resistance approach, Starossek proposes that a primary load transfer system being a key element could take the form of a massive tube as illustrated in Fig. 4.8. The tube core should constitute a high-strength reinforced concrete wall or a steel shape embedded in reinforced concrete with the wall thickness of the order of 1 m or more. As seen in this Figure, the tube core should not be situated on the outer perimeter of the building, because any openings in the core should be limited to the minimum. There are also
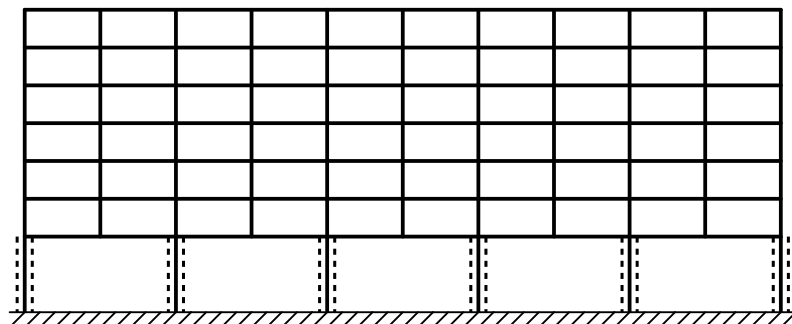
**Figure 4.4.** Protective barriers
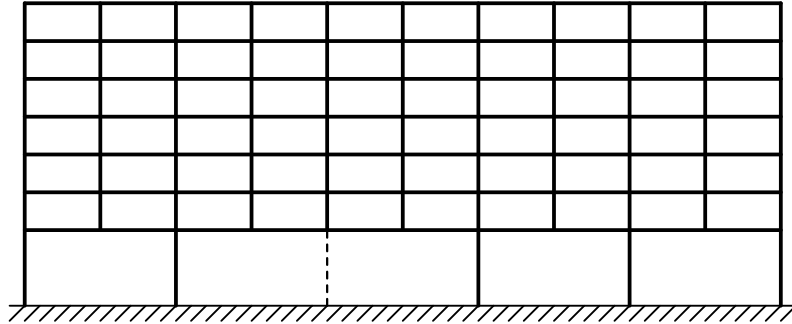
Figure 4.5. Column failure
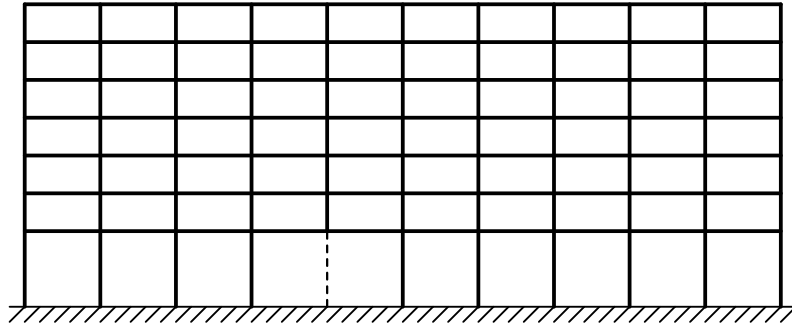
**Figure 4.6.** Modification of structural system - more columns

other features which the tube core should possesses that are described thoroughly in the paper. In addition to the primary load transfer system there is envisaged a secondary load transfer system in a form of cantilever floors fixed to the core. These cantilever floors, on the other hand, should be designed according to the alternate load path method, and any local failure should not lead to the partial collapse. Allowable damage scenarios of the secondary load transfer system are presented in Fig. 4.9. It should be noted that an adequate rotational ductility capacity of the plastic hinges should be ensured. Starossek states that the required rotational ductility capacity is difficult to be achieved in reinforced concrete beams and instead, haunched steel girders can be used. Other way to limit the consequences of a local failure is to design segmentation of the secondary load transfer system by means of joints as illustrated in Fig. 4.10. Concerning the alternate load paths approach for the primary load transfer system, Starossek concludes that it is almost impossible to achieve for tall buildings nowadays, giving the example of the total collapse of the WTC towers. On the other hand, using vertical segmentation approach, there are chances that an initial local failure can be arrested. An example of such vertical
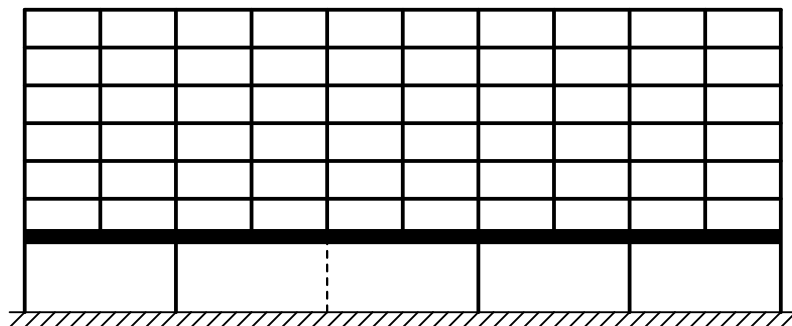

Figure 4.7. Strengthening of transfer girders
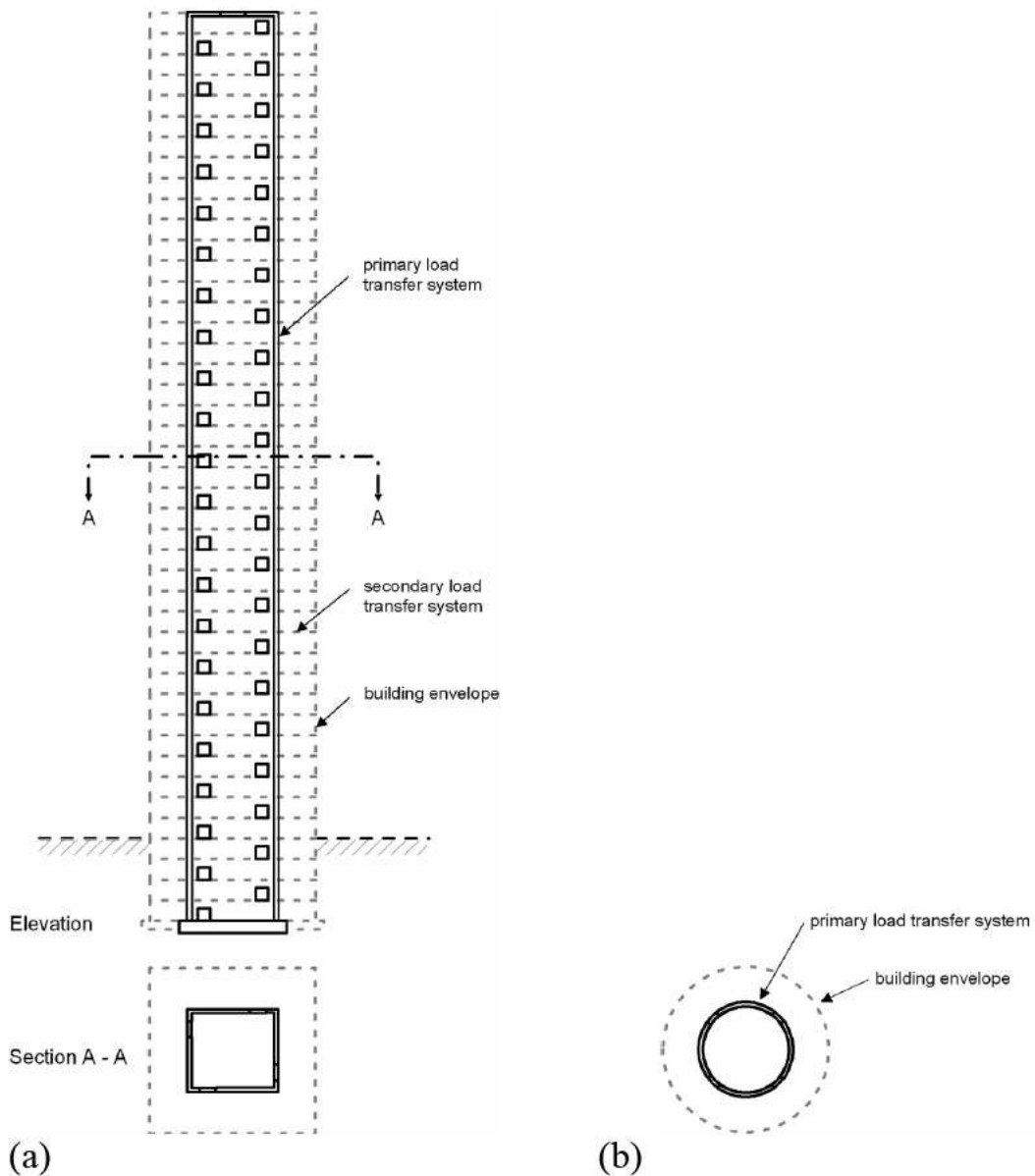
Figure 4.8. Primary load transfer system: a) elevation and rectangular cross-section, b) circular cross section (from Starossek [67])
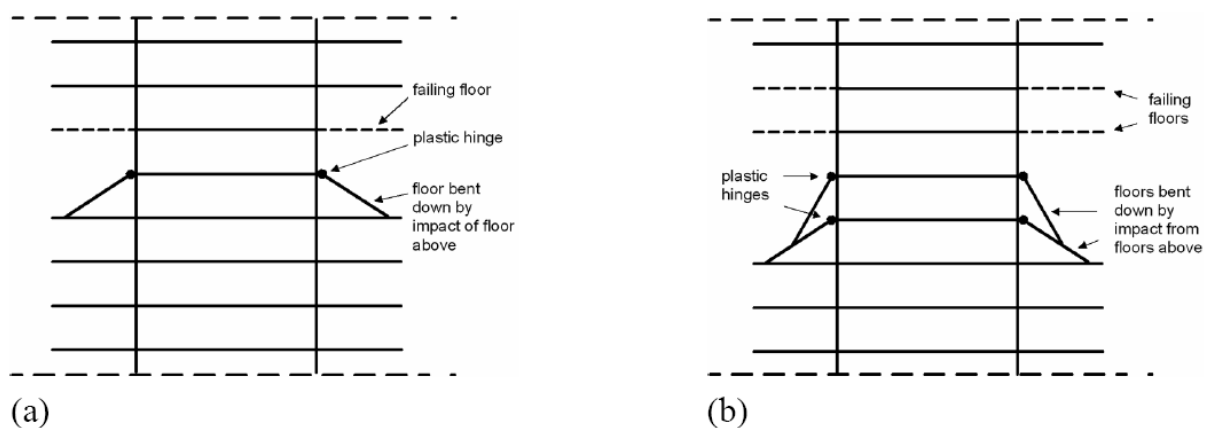

Figure 4.9. Assumed damage and admissible deformation in secondary load transfer system: a) assuming the impact of one floor on another floor below, b) assuming the impact of two floors on another two floors below (from Starossek [67])
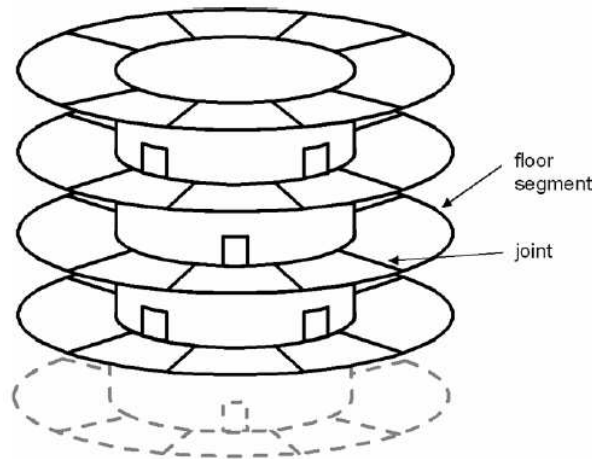
Figure 4.10. Segmentation of secondary load transfer system by joints (from Starossek [67])

segmentation in a tall building is given in Fig. 4.11. The height of a segment should be of the order of one tenth of the building height. It is assumed that in the case of a local failure of one or more floors, the failure front progresses vertically in two directions (upward and downward) of segment borders. Simultaneously, the upper part of the building, above the failing segment, moves down as a rigid body. The largest forces occur when the upper part together with the falling debris impacts the lower segment. First solution could be to design thick reinforced or prestressed concrete slabs. The other more advanced solution would comprise two slabs having shock-absorbing devices in between (see Fig. 4.11). These envisaged shock-absorbing devices could consist of telescoping large diameter steel tubes filled with a material which enables high compressive strain, for example using scrap metal or porous tuff gravel (see Fig. 4.11c). Regarding the prescriptive design rules for tall building, Starossek advises against using them since for such large and expensive structures general rules without performance-based analysis can enhance the continuity, ductility and catenary actions in a quantitative way but not in a qualitative one.

DeStefano [17] as a practitioning engineer gives recommendations on how to provide adequate detailing to prevent progressive collapse without excessive costs. He takes advantage of the fundamental principle of catenary behaviour which can be developed when structural elements originally designed to resist loads in flexure are subjected to large displacements which in turn causes to develop axial forces in those structural elements. For instance when an exterior column is removed from the lower floor of the structure, the ring girders act in tension. However, the weakest point in this consideration is the corner of the building. The author of the paper provides a simple solution by applying diagonal girder in each corner span. There are also given four tips on how to design the structure to allow for catenary behaviour. These are: (1) orientation of girders in such a way that the heaviest members are on the perimeter, (2) introduction of diagonal girders in the corner spans, (3) orientation of wide-flange perimeter columns with their strong axis parallel to the exterior wall. This allows the girders to connect directly to the column flange, (4) use of ductile girder connection to the columns to allow rotations and to resist large axial loads. Seat and one-sided connections should be avoided.
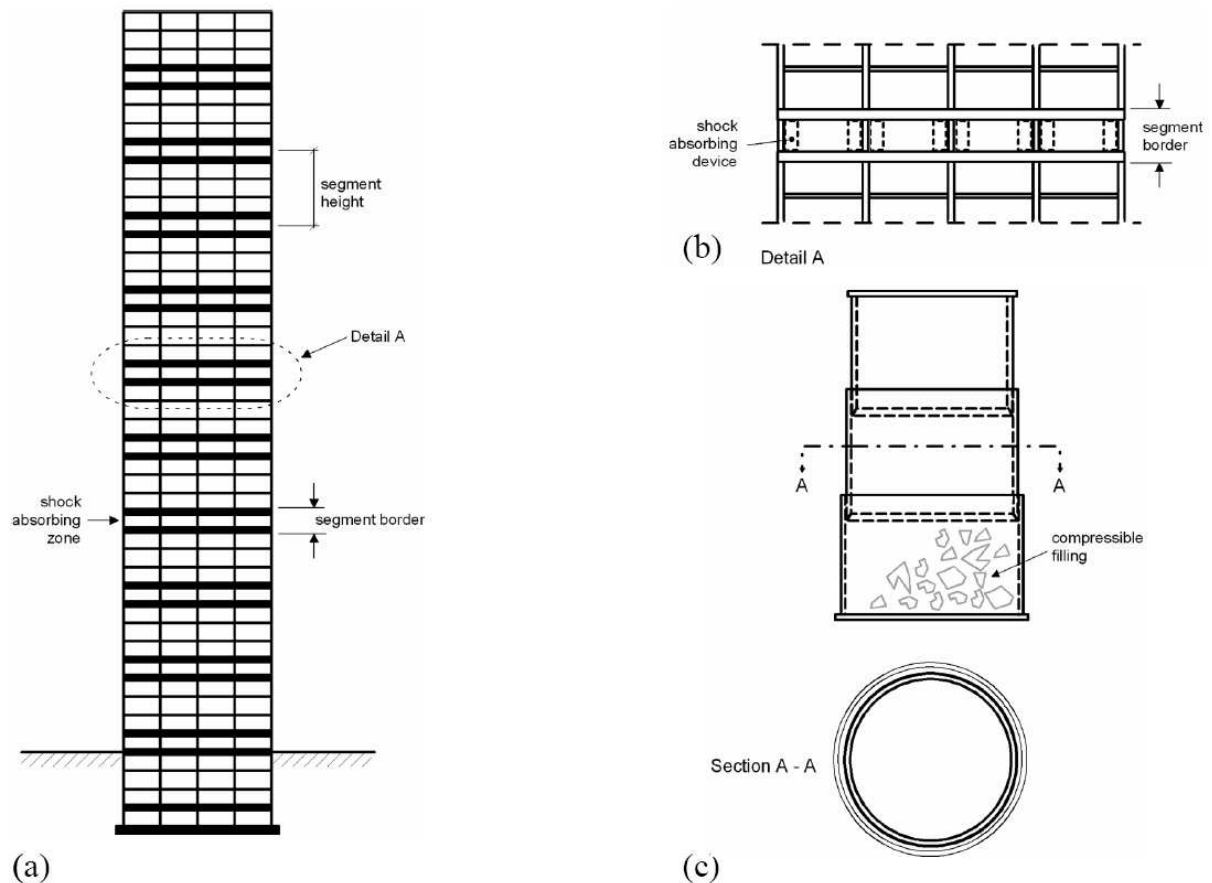
Figure 4.11. Segmentation strategy: (a) overall view of a building with vertical segmentation, (b) detail of the shock-absorbing segment, (c) shock-absorbing device: telescoping steel tubes with compressible filling (from Starossek [67])

Another interesting example of progressive collapse design given in the same paper by Starossek and Wolff [70] is the approach in the case of the Confederation Bridge in Canada. Designing this kind of bridge in a robust way appeared not to be feasible, so the actual approach for progressive collapse was to spatially limit potential local failure by isolating the collapsing section. Avoiding potential total collapse of the bridge required an interruption of continuity of prestressed tendons.

An interesting case of removing columns experimentally in an existing building was presented by Sasani and Sagiroglu [62]. The building was the 6-storey Hotel San Diego, San Diego, U.S., planned to be demolished. The height of the first floor was about 6 m, while the height of the other floors and the last floor were 3.2 m and 5.13 m, respectively. The structural system constituted a non-ductile reinforced concrete frame structure with exterior infill walls. The floor system consisted of one-way joists. Before the experiment, all nonstructural elements, infrastructure and furniture as well as the infill walls of the first and the third floor were removed deliberately. The plan, south-east view and a 3D model of the Hotel is in Fig. 4.12, 4.13 and 4.14.

To evaluate progressive collapse resistance of the building, two columns of the first floor (including a corner column) were exploded by implosion. The explosives were placed in drilled holes in the columns and the columns were wrapped

Figure 4.12. South-east view of the Hotel San Diego (from Sasani and Sagiroglu [62])
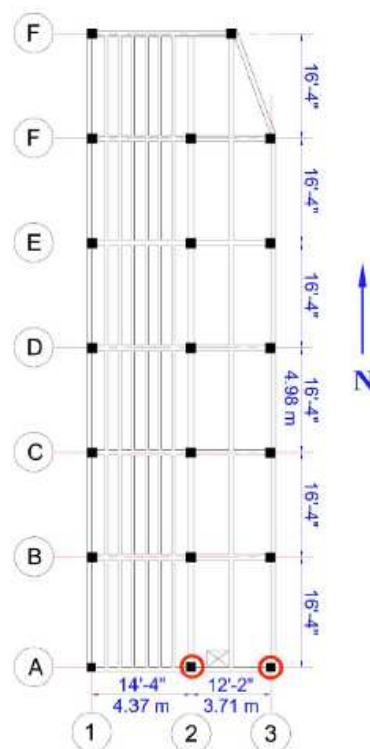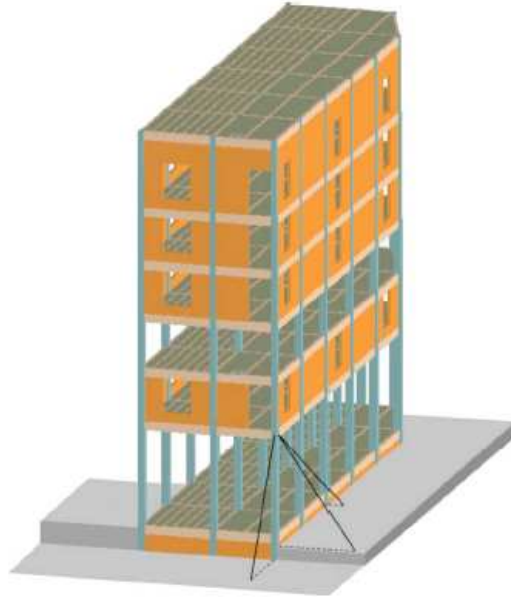

Figure 4.13. Typical plan of the Hotel San Diego (removed columns in circles) [62]

Figure 4.14. 3D model of the Hotel San Diego (from Sasani and Sagiroglu [62])

by protective materials what prevented from air-blast shock waves. The structure was equipped with strain gauges and potentiometers to measure global and local deformations in time. The actual sudden removal of the two columns didn't cause any collapse of the frame. The maximum and the permanent vertical displacements at points above the removed columns were around 6.1 mm, which means that the beam above the removed columns deflected as a rigid body without deformations. On the other hand, the beams A1-A2 and A3-B3 deformed more significantly with rotations illustrated in Fig. 4.15. The reproduced deformed shape of the frame based on measurements is shown in Fig. 4.16. It is interesting to note that beams A1-A2 and B3-A3 exhibits double curvature which accounts for the development of Vierendeel frame action in both directions (see also Fig. 4.16)

Abruzzo et al. [1] report the application of progressive collapse procedures to assess an existing reinforced concrete commercial building. The applied progressive collapse procedures were: ACI 318 [2] structural integrity requirements, tie force approach based on DoD UFC Guidelines [20], linear static method based on GSA Guidelines [40] and nonlinear static analysis based on DoD UFC Guidelines [20] (using SAP2000). The authors state that the structure would be prone to progressive collapse when one of the interior columns loses capacity to carry loads. In particular, prescriptive requirement such as those in ACI 318 [2] and DoD UFC Guidelines [20] would not guarantee the prevention of progressive collapse.
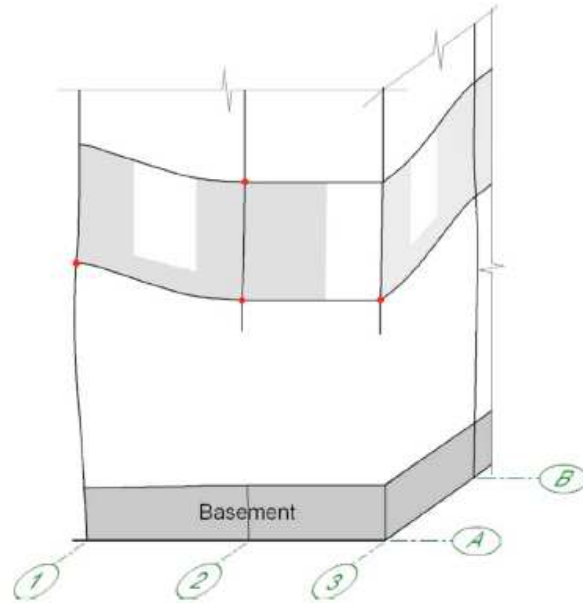
4.4. Types of progressive collapses

Starossek [66] in his paper presents six different types and four classes of progressive collapse mechanisms. Those mechanisms are described by their characteristic features and then are compared. The motivation for this typology and classifica-
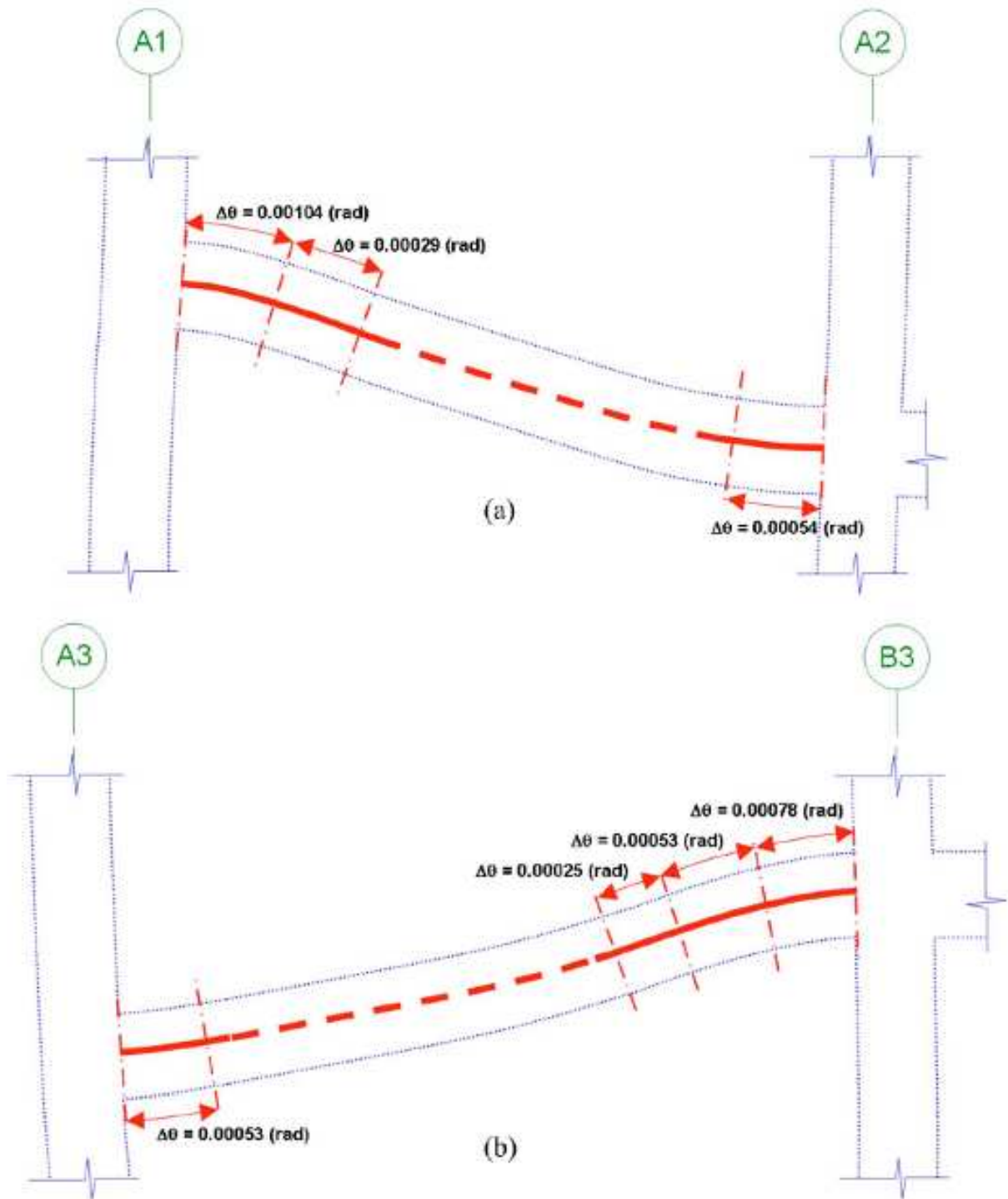
Figure 4.15. Deformed shape of beams A1-A2 and A3-B3 (from Sasani and Sagiroglu [62])

Figure 4.16. Reproduced deformed shape after columns removal based on measurements (from Sasani and Sagiroglu [62])

tion is that although the progressive collapse is understood as failure of the whole system due to a disproportionately small local failure, the final collapse state can be produced in various ways. The author of this paper states that little progress in developing analysis procedures, quantifying indices (e.g. such as robustness indices) and developing, classifying and choosing countermeasures might have been achieved due to the lack of differentiation and classification of progressive collapse types. Below the list of types of progressive collapse with their main features is presented, followed by the classification and proposed terminology:

1. Pancake-type collapse.
   A pancake-type collapse is characterised by the following features:
   - initial failure of load-bearing members;
   - partial or complete separation and fall of components, in a vertical rigid-body motion;
   - transformation of potential energy into kinetic energy;
   - impact of separated and falling structural components on the remaining structure;
   - failure of the vertical load-bearing elements due to the impact loading;
   - collapse progression in the vertical direction.
   Example: the collapse of World Trade Center towers.
2. Zipper-type collapse.
   A zipper-type collapse is characterised by the following features:
   - initial failure of one or a few structural elements
   - redistribution of forces carried by these elements in the remaining structure
   - impulsive loading due to the suddenness of the initial failure
   - dynamic response of the remaining structure to that impulsive loading
   - due to the combined static and dynamic effects, a force concentration in and failure of elements which are similar in type and function to and adjacent to or in the vicinity of the initially failing elements

- collapse progression in a direction transverse to the principal forces in the failing elements

Example: the collapse of Tacoma Narrows Bridge.

3. Domino-type collapse.

   The list of features characterising a domino-type collapse is as follows:
   - initial overturning of one element
   - fall of that element in an angular rigid-body motion around a bottom edge
   - transformation of potential energy into kinetic energy
   - abrupt deceleration of the element's motion through sudden activation of discrete other elements; the horizontal force induced by that event is of both static and dynamic origin because it results from both the tilting and the motion of the decelerated element
   - overturning of other elements due to the horizontal loading from the decelerated element
   - collapse progression in horizontal direction

4. Section-type collapse.

   It can be described in the context of a beam in bending or a bar under axial tension. When a cut occurs in a section of the beam, the internal forces must be transmitted by the remaining cross-section. This type of collapse can not usually be called a progressive collapse but fast fracture. However it is included in the typology list to exploit similarities and analogies.

5. Instability-type collapse.

   An instability-type collapse is characterised by the following features:
   - initial failure of elements which stabilise load-carrying elements in compression
   - instability of the elements in compression that cease to be stabilised
   - sudden failure of these destabilised elements due to small perturbations
   - failure progression

6. Mixed-type collapse. Other types of progressive collapse mechanisms which cannot be easily distinguished and described and/or can be a combination of the previous five types, fall in the mixed-type progressive collapse. As an example of this type of progressive collapse, Starossek [66] gives the partial collapse of the Alfred P. Murrah Federal Building (Oklahoma City, 1995) and Haeng-Ju Grand Bridge (Seoul, 1992).

Those types of progressive collapse can be further classified by their main features. So zipper-type and section-type collapses can be grouped into the redistribution class since the remaining structure must redistribute the forces of failed elements. Pancake-type and domino-type collapses are characterised by the fact that the major amount of potential energy is transformed into kinetic energy. So they can be grouped into the impact class. At last, instability-type and mixed-type collapses form their own classes having no common features.

4.5. Simplified methods for progressive collapse analysis

Buscemi and Marjanishvili [13] present a simplified method for assessing the predisposition of a structure to progressive collapse. The method is based on pendulum analogy, reducing the problem to a dynamic analysis of a SDOF system with initial conditions. The authors developed a modelling aid which can be used to determine how a building is prone to progressive collapse based on maximum ductility and corresponding rotations of major load-bearing elements.

Pujol and Smith-Pardo [59] present a simplified method for analysing frame structures for progressive collapse when a column is instantaneously removed. The purpose of the method is to provide a simple tool for preliminary design and consists in evaluating a displacement ductility demand (ratio of displacement demand to yield displacement – $\mu = \Delta_{max}/\Delta_y$) using load factors (ratio of yield resistance to applied load – LF). The authors conclude that a floor system will have enough progressive collapse resistance when one or more supports are removed provided that: (1) the strength of the floor system, after the removal of support(s), is greater than the applied loads by more than 50% and (2) the displacement ductility capacity of the floor system is greater than 1.5. For other ratios of yield resistance to applied load (LF), the chances of the floor system to survive depends on displacement ductility demand (μ) which can be estimated using the following formula

$$\mu = \frac{1}{2} \cdot \frac{LF}{LF - 1}. \tag{4.2}$$

Menchel [52] devoted his Ph.D. thesis to progressive collapse. The main contribution is the development of two new quasi-static procedures accounting for dynamic inertial effects more accurately. The first new pushover analysis procedure is based on a kinetic energy criterion (similar approach can also be found in Izzuddin et al. [43] and Dusenberry and Hamburger [23]) and the second pushover analysis is based on optimised load amplification factors. The validation of the procedures were performed using dynamic nonlinear analyses. Another contribution is to use a more complex fibre-based nonlinear beam element (instead of an elastic beam element with discrete plastic hinges) to analyse reinforced concrete beams and columns.

Dusenberry and Hamburger [23] introduced a new energy based progressive collapse analysis method which creates a bridge between present simplified analysis methods and the best representation of the nonlinear dynamic behaviour that occurs during collapse. The energy-based method tracks the energy released by falling weight and energy absorbed by the structure, a comparison can be made to assess if the failure is arrested. In other words, if the energy absorbed by the structure exceeds the change in potential energy, the structure comes to rest and has potential to survive.

Vlassis et al. [74] present a new design-oriented methodology for progressive collapse assessment of multi-storey buildings subjected to impact from the above failed floor. The idea is to assess if the lower floor is able to arrest the falling floor

which depends on the amount of kinetic energy transmitted from the upper floor during the impact.

Schmidt [63] provides a summary of the methods which enable to define an external terrorist bomb threat, estimate structural design loads and analyse the structure using a simple dynamic model. The effect of a blast is defined through the parameter $Z = R/W^{0.333}$, where R is a stand-off distance and W is explosive charge size. The stand-off distance measures how close to the structure, a bomb can explode and therefore is related to the physical surroundings of the building. W is a corresponding mass or weight of TNT. For other explosives, an adequate scaling factor relating the heat energy of detonation to that of TNT must be provided. As for blast loading, the shock wave caused by an external blast gives an almost instantaneous increase in pressure on objects to a maximum value. After reaching the maximum of the positive phase, the pressure decays and then for longer period rests in negative phase. For structural analysis, this behaviour can be approximated by a triangular impulse load with zero or very small rise time and longer linear decay. The maximum reflected pressure (p_r) and total reflected impulse (i_r) of the actual load in positive phase should be the basis for the equivalent loading, so the design duration be $t_d = 2i_r/p_r$. The negative phase is neglected because it has little effect on the maximum response of the structure.

4.6. Numerical case studies

Bao and Li [8] analysed the residual strength of reinforced concrete columns under a short stand-off blast loading. A case studies were performed to evaluate the effects of transverse reinforcement ratio, axial load ratio, longitudinal reinforcement ratio and column aspect ratio using the explicit FEM software LS-DYNA. In the paper the effects of strain rate on concrete and reinforcing steel were taken into account. The numerical simulation of dynamic response revealed that:

- the same axial load ratio is more critical for columns with a low transverse reinforcement ratio,
- the ratio of residual axial capacity increases with the longitudinal reinforcement ratio,
- the residual axial capacity increases with the decrease of the column aspect ratio,
- seismic detailing techniques can significantly reduce the extent of direct blast induced damage.

Agnew and Marjanishvili [4], Marjanishvili and Agnew [49], Marjanishvili [50] present and compare four methods for progressive collapse analysis by analysing a 9-storey steel moment-resistant frame building. The four methods are: linear static, nonlinear static, linear dynamic and nonlinear dynamic methods. The last paper gives detailed steps for performing all analyses in SAP 2000 including some screenshots. Contrary to the GSA Guidelines and DoD UFC guidelines, where they discourage to use nonlinear dynamic analysis, the authors of the papers state that

using nonlinear dynamic analysis is not only more accurate but also easy to perform by using modern FEM software.

Kaewkulchai and Williamson [44, 45] presented a nonlinear solution procedure for progressive collapse analysis of planar frame structures. A proposal of a modelling strategy to account for the impact of failed members against other structural components is given. The authors indicate that the impact velocity plays the most important role.

Ruth et al. [61] performed analyses which lead to conclusion that the dynamic factors of 2 given in GSA Guidelines [40] and DoD UFC Guidelines [20] are too conservative. Comparison of an equivalent static nonlinear analysis with the dynamic factor of 1.5 and the dynamic nonlinear analysis shows that enough safety is ensured and more economic design can be achieved.

Powell [58] also confirms that the static analysis for progressive collapse is too conservative, also stating that performing dynamic, nonlinear analysis using current FEM software is no more difficult and gives better accuracy.

Hayes et al. [41] presents studies on how the current seismic design provisions can improve resistance to blast loads and progressive collapse. The study was carried out on Alfred P. Murrah Federal Building (severely damaged in 1995), where 3 seismic strengthening techniques and then progressive collapse analysis were performed to compare progressive collapse resistance of the building. The key finding of the study was that strengthening the perimeter element using current seismic detailing techniques improved the survivability of the building, while strengthening elements internal to the building was not effective.

4.7. Measures of structural robustness and vulnerability

According to NIST Best Practices [56], structural robustness for reducing progressive collapse risk can be enhanced by incorporating sufficient: redundancy, ties, ductility, adequate shear strength and capacity for resisting load reversals. Although providing adequate level of these features will improve the robustness of the structure qualitatively, it would be convenient to use a quantitative way to measure and compare the robustness of different structures.

Starossek and Haberland [68, 69] formulated four general conditions that measures of robustness should satisfy (if possible). These general conditions are:

- expressiveness: the measure should express all important aspects of robustness and should give a clear difference between robust and non-robust structures;
- objectivity: the measure should be independent of decisions and same results should be obtained for same conditions;
- simplicity: the measure should be as simple as possible;

- calculability: the necessary parameters should be quantifiable and should be obtainable from the data of the structure;
- generality: the measure should be possible to apply for different structures.

Different measures of robustness found in literature can be divided on those based on structural behaviour and those based on structural attributes.

4.7.1. Measures based on structural behaviour

Starossek and Haberland [68, 69] proposed three measure of robustness. Two measures of robustness fall into this category of measures based on structural behaviour and the third one is based on structural attributes. The first measure of robustness proposed by the authors is based on damage and defined as

$$R_d = 1 - \frac{p}{p_{\lim}},\tag{4.3}$$

where R_d – damage-based robustness measure, p – maximum extent of additional damage (maximum damage progression) caused by the assumed initial damage $i_{\lim}$, $p_{\lim}$ – acceptable damage progression. This measure takes into account design objectives such as assumed initial damage and acceptable damage progression. Another version of this measure of robustness can be expressed as

$$R_{d,int} = 1 - 2\int_0^1 [d(i) - i]di,\tag{4.4}$$

where $R_{d,int}$ – damage-based integral robustness measure, $d(i)$ – maximum extent of total damage caused by and including the initial damage i, based on the corresponding value for the intact building, i – extent of initial damage also based on the intact building.

The illustration of damage progression is shown in Fig. 4.17, where curve A represents a non-robust structure showing that even small initial damage can lead to large global damage; curve B represents a robust structure – significant damage occurs when large initial damage happens. However similar areas between the curves B and C and line i (for integral in Eq. (4.4)) can be misleading in determining a good robustness measure.
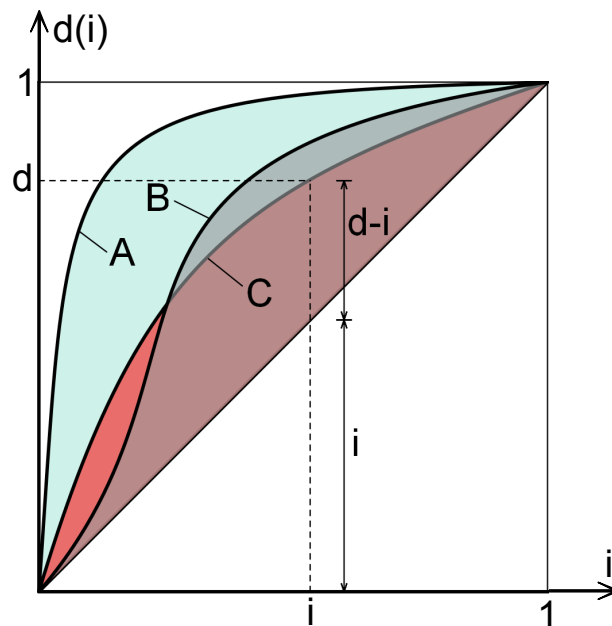

Figure 4.17. Damage evolution

The second measure of robustness, based on energy, presented by Starossek and Haberland [68, 69], can be defined as

$$R_e = 1 - \max_j \frac{E_{r,j}}{E_{s,k}}, \tag{4.5}$$

where R_e – energy-based robustness measure, $E_{r,j}$ – energy released by the initial failure of the structural element j and available for the damage of the next structural element k, $E_{s,k}$ – energy required for the failure of the next structural element k.

Lind [46, 47] presents probabilistic definition of vulnerability and damage tolerance (robustness) and their related measures. The suggested definitions are intended to be general and applicable to all engineering systems. To define vulnerability, Lind denotes $P(r, S)$ as the probability of failure of the system in a state r for prospective loading S, r_0 as a pristine system state and r_d as a particular damaged state. Then the vulnerability is defined as the ratio

$$V = V(r_d, S) = \frac{P(r_d, S)}{P(r_0, S)}. \tag{4.6}$$

This vulnerability is equal to one if the probability of failure is the same in the damaged and undamaged states. The measure of damage tolerance is defined as the reciprocal of the vulnerability (Eq. (4.6))

$$T = \frac{P(r_0, S)}{P(r_d, S)}. \tag{4.7}$$

As the author states, to have realistic measure of vulnerability and damage tolerance, one needs to assign realistic probability distributions for them.

Baker et al. [7] proposed a framework for assessment of robustness for systems subject to structural damage. The idea of this framework is to create an event tree as follows. Assume that a certain potentially damaging event (e.g. extreme values of snow loads, explosions etc.) occurs in the system. The system can be damaged or not. If no damage occurs, no further analysis is needed. If damage occurs, different damage states can happen. For each damage states assign a probability of failure. Assign also consequences for possible damage and failure scenarios. Direct consequences are related to initial damage, whereas indirect consequences are related to subsequent system failure. Consequences take multiple forms for example: inconvenience to system users, injuries, fatalities and costs and require an introduction of a scalar measure. Having assign all these values, the authors proposed an index of robustness (which measures the fraction of the total system risk resulting from direct consequences) as follows

$$I_R = \frac{R_{Dir}}{R_{Dir} + R_{Ind}}, \tag{4.8}$$

where R_{Dir} and R_{Ind} are direct and indirect risks, respectively. These risks are calculated by multiplying the consequences of each possible event scenario by its probability of occurrence and then integrating over all the random variables in the event tree. There are in the paper some example calculations for idealised systems, however as the authors state application for real system requires more work.

Wisniewski et al. [75] report load-capacity evaluation of existing railway bridges by robustness quantification. Bridge robustness is defined as the ability to carry loads after the failure of one of its members. To express robustness of railway bridges, the authors define so-called redundancy factor based on redundancy ratios which compare the load-carrying capacity in the limit states to the design load-carrying capacity. If the calculated redundancy factor is less than 1.0 then the bridge may be deemed as not safe, for redundancy factor equal or greater than 1.0 the bridge may be considered as safe.

Maes et al. [48] presented three measures of robustness. The first measure of robustness is referred to the assumption that the system's resistance as a whole can be maintained to a sufficient degree after a load that makes failure in one of its components i. This measure of robustness can be written as

$$R_1 = \min_i \frac{RSR_i}{RSR_0},\qquad(4.9)$$

where RSR_i is the reserve strength ratio when member i is impaired, while RSR_0 is the reserve strength ratio when no member is impaired.

The second proposed measure of robustness can be expressed in terms of system reliability as

$$R_2 = \min_i \frac{P_{s0}}{P_{si}},\qquad(4.10)$$

where P_{s0} is the system probability of failure of the undamaged system subject to such a member design load and P_{si} is the system probability of failure when element i is impaired.

The third measure of robustness is based on sample functions of failure consequences versus hazard intensity and conditional probability of exceedance versus failure consequences. Computing the inverse of the tail heaviness H of the mentioned conditional probability of exceedance versus failure consequences a measure of robustness R_3 can be obtained. According to Maes et al. [48], the tail of any given probability distribution can be easily computed and if H is less than one, it means that the robustness of a structure is very high, if H is one – the robustness is smaller than before and for large H – the robustness of the structure is very low.

Smith [64] proposed a measure of robustness based on the theory of fast fracture in fracture mechanics. The premise of this approach is as follows: if the energy released by loss of a damaged member is greater than the energy absorbed by the totally damaged member and other partially damaged members, then the progressive collapse will occur. This approach uses finite element analysis of a structure and graph theory search to find the sequence of damage events that requires the smallest amount of energy. The minimum damage energy is a measure of robustness of a structure.

Menchel [52] introduced an indicator of robustness of a structure with respect to the removal of a column defining it as the maximum load factor that can be applied to the dead and live loads when applied statically on the structure from which the

considered column has been removed. The procedure to estimate such an indicator of robustness is as follows: (1) remove a column from the model, (2) apply the loads statically, multiplying them by an increasing factor, (3) when stresses can no longer be redistributed, this factor is the indicator of robustness. In other words, the indicator of robustness is a measure of the resistance reserve of a structure with regard to the loads applied to it and for the removal of a specific column.

4.7.2. Measures based on structural attributes

As mentioned before the third measure of robustness defined by Starossek and Haberland [68] falls into the group of measures based on structural attributes, here specifically on structural stiffness and can be expressed as

$$R_s = \min_j \frac{\det \boldsymbol{K}_j}{\det \boldsymbol{K}_0} \tag{4.11}$$

where R_s - stiffness-based robustness measure, $\boldsymbol{K}_0$ and $\boldsymbol{K}_j$ – global stiffness matrix of the intact structure and the structure after removal of a structural element or a connection j, respectively. The authors state that there is a relatively low correlation between the reduction in load capacity after the loss of structural element and the assigned measure of robustness R_s. However this formulation has two advantages namely: simplicity and ease of calculation. This approach can be appropriate for zipper-type collapse and useless for pancake-type or domino-type collapses as defined in [66] and summarised above in Section 4.4.

Agarwal et al. [3] introduced a topological measure of vulnerability referring to the theory of form and connectivity. Basically it investigates the potential hazards present in the structural form which could be triggered by unexpected events or unforeseen loading conditions. The theory consists of three steps which are: identifying so called structural rings and rounds, creating a hierarchical description of the structure and searching through the structural hierarchy to look for vulnerable scenarios. This approach is developed for 2D and 3D frames.

4.8. Other topics

Tagel-Din and Rahman [72] present a new progressive collapse simulation method based on the Applied Element Method (AEM). The method takes into account load thread, separation of structural parts and possible collisions resulting from falling debris. In the Applied Element Method, a structure is modelled as an assembly of small elements connected be a series of springs. Partial connectivity occurring during progressive collapse is allowed and modelled by means of springs. When modelling a structure, all that is required is to define the material type, or property of different interfaces.

Moore [54] presents how provisions in British Standards defined after the Ronan Point collapse improved the situation in the UK. Moore states that engineers mainly

used indirect approach (tie force requirements). In addition, the author proposes to extend the application of the current UK building regulations to all buildings, and not only to 5 or more storey buildings.

Report NIST Best Practices [56] gives the state-of-the-art on progressive collapse, based on British Standards, ASCE, EC and other national and local code standards as well as based on GSA Guidelines, DoD UFC guidelines. The main topics included in the report are: consideration acceptable risk, building vulnerabilities (continuity, load reversals), means of risk reduction (event control, structural analysis), detailed description of indirect and direct approaches, extensive comparison of code, standards and national guidelines. The document summarises best practices for designing different type of structures (r/c, steel, masonry etc.).

In 2011 the COST Action TU0601 group concluded their work on robustness of structures in [14]. The purpose of this document is to provide additional information to designers, structural analysts, regulatory authorities etc. The document is also aimed at providing assistance to the committees developing codes and standards (e.g. CEN TC250 and ISO TC98). The particular aspects dealt in this document contain:

- failure of more than one load-bearing element, by considering the various types of loading,
- assessment and quantification of robustness (most of the robustness indices are described in Chapter 4.7),
- risk-based decision-making,
- methods for providing robustness (e.g. specific local resistance method, alternate load path method, tie force method etc.),
- robustness during construction,
- effects of poor quality and deterioration of material.

In 2011 US Department of Homeland Security [73] issued comprehensive technical report on how to prevent structures from collapsing to limit damage to adjacent structures and additional loss of life when explosives devices impact highly populated urban centers. The main focus is on:

- the influence of the presence of buildings on the blast pressures propagating from explosions located in urban settings,
- the potential for these blast pressures to damage primary structural members of buildings,
- the sensitivity of several common building design types to experience progressive collapse due to damage of key support members, and
- the likelihood that blast pressures may damage building equipment needed for emergency evacuation, rescue and recovery operations.

The results the studies were integrated in the prototype of a fast running software program, concentrated on the Manhattan Financial District.

Stevens et al. [71] .

5. Examples of progressive collapse

This chapter presents selected examples of building progressive collapses. Although the number of progressive collapses in the history is quite small, the catastrophic consequences in terms of fatalities and other losses, which this phenomenon entails, brings a lot of attention in society, governments and community of civil engineers. Moreover, the growing threat of terrorist attacks makes the problem of progressive collapse should be considered when designing and constructing buildings.

5.1. Ronan Point

The Ronan Point Apartment building was erected in London between 1966 and 1968. It was a 23 storey, and 64 m high building. The structural system consisted of precast concrete walls and floors. The floors were supported by the lower storeys walls. The floors and walls were fitted by slots and bolted together. The connections were filled with dry packed mortar. Thus, this structural system was characterised by very limited ability to redistribute loads and was prone to progressive collapse when exposing to a local failure.

On May 16, 1968, an exterior panel of the 18th floor was blown out due to an internal gas explosion of a leaking gas stove. The loss of this panel caused the progressive collapse of upper floors (19 through 23) and subsequently the falling debris invoked the progressive collapse of lower floors down to the ground floor. The result of the progressive collapse can be seen in Fig. 5.1.

More details can be found in [56].

5.2. Alfred P. Murrah Federal Building

The Alfred P. Murrah Federal Building located in Oklahoma City was an office building of the U.S. government constructed between 1970 and 1976. It was a nine-storey, reinforced concrete structure, 61 m long, 21.4 m wide. The structural system was formed by columns (on 6.1 m × 10.7 m grid) and a beam/slab system. The regularity of the frame was interrupted on the north elevation where the span between columns was doubled and these columns supported deep transfer girders.

Figure 5.1. Ronan Point Apartment after collapse [16]

It should be noted that the reinforced concrete frame was designed and constructed in a non-ductile manner.

On April 19, 1995, the Alfred P. Murrah Federal Building became the target of a terrorist attack. The explosive charge was detonated from a truck situated 4 m from one of the columns. The power of the blast was estimated to 1800 kg of TNT equivalent (see Osteraas [57]). The direct blast destroyed one column, then the blast wave destroyed floors and beams which in turn was the cause for buckling the other three columns due to lack of lateral supports. Two illustrations of the Alfred P. Murrah Federal Building after the progressive collapse can be seen in Fig. 5.2 and 5.3.

After investigating the Alfred P. Murrah Federal Building case, Osteraas [57] concludes with four statements, which should be applied in progressive collapse design, namely:

- avoiding significant irregularities in the layout configuration and using transfer girders, to ensure the possibility of alternative load paths,
- the frame should enable the slabs and walls to fail without destroying the frame,
- the frame should be robust and ductile to sustain large deformations,
- lower perimeter columns should be designed to resist direct blast waves as much as possible.

The author states that the third conclusion can entail adopting detailing from seismic engineering and applying it for designing important buildings in nonseismic zones.

Figure 5.2. Alfred P. Murrah Federal Building [60]

Figure 5.3. Alfred P. Murrah Federal Building (Note the brittle failure of the transfer girder due to non-ductile design) [15]

Further details can be found in [56, 57].

5.3. L'Ambiance Plaza

L'Ambiance Plaza in Bridgeport, Connecticut, was a 16-storey apartment building which collapsed in April 1987 while being under construction (see Fig. 5.4). The building was erected using lift-slab method, which required the slabs to be cast on the ground and then lifted into place using jacketing operations. The total collapse happened after finishing one of the jacketing operations (see Fig. 5.5). After the investigations of the collapse by Heger [42], there were reported four main structural deficiencies which could cause the collapse, namely: (1) improper placement of post-tensioning tendons adjacent to elevator openings, (2) overstressed concrete slab sections adjacent to two temporary floor slots for cast-in-place shear walls, (3) overstressed and excessively flexible steel lifting angles during slab lifting, and (4) unreliable and inadequate temporary slab-column connections to ensure frame stability. On the other hand, Dusenberry [22] states that possible causes that initiated

Figure 5.4. L'Ambiance Plaza during the construction phase [22]

the collapse were: failure of shear collars, improper configuration of tendons, overall stability, plumbing operation, lateral soil pressure and frozen concrete, while the direct reason for the complete collapse could be the use of a very mild steel and incomplete construction.

More details can be found in [42, 51, 56].

Figure 5.5. L'Ambiance Plaza after the collapse [22]

6. Summary and conclusions

The aim of this report is to review the state-of-the-art of progressive collapse procedures and strategies provided in codes and standards as well as in selected national specific guidelines (e.g. DoD UFC Guidelines [20], GSA Guidelines [40]). Recent growing interest in the subject of progressive collapse of civil engineering structures, resulted in an observed increased number of scientific publications. Findings from several important papers are also reviewed in Section 4.

To summarise the provisions raised in the studied documents, selected aspects are presented in the following tables. A comparison of load combinations for progressive collapse analysis in the different documents is presented in Table 6.1.

Table 6.1. Comparison of load combinations in different documents

Document	Load Combinations	Remarks
BS	$D + L/3 + W/3$	
Eurocode	see Eq. (3.5)	
ASCE 7-05	$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W_n$	alternate load path method
	$1.2D + A_k + (0.5L \text{ or } 0.2S)$	specific local resistance m.
	$(0.9 \text{ or } 1.2)D + A_k + 0.2W_n$	specific local resistance m.
GSA	$2(D + 0.25L)$	static analysis,
	$D + 0.25L$	dynamic analysis,
UFC 4-023-03	$(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W_n$	nonlinear dynamic analysis
	$2[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S) + 0.2W_n]$	nonlinear static analysis

D – dead load,
L – live load,
W – wind load,
S – snow load.

A comparison of accidental loads for designing key elements in the different documents is presented in Table 6.2. It should be noted that the Eurocodes inherited the value of $34\,\text{kPa}$ from the British Standards. In many places, however, this value is questioned as being too high. On the other hand, other documents presented in the table do not specify the value leaving it to the authorities. The U.S. guidelines, overall, prefer the alternate load path method instead of the specific local resistance method, and do not mention accidental loads at all.

Table 6.3 presents a comparison of allowable local collapse areas in the selected documents. These limitations are similar, however DoD UFC Guidelines [20] distinguishes the limitations depending on where the local damage takes place.

Table 6.2. Comparison of accidental loads for designing key elements in different documents

Document	Accidental load
BS	34 kPa
Eurocodes	34 kPa
ASCE 7-05	A_k (to be specified by the authorities)
GSA	-
UFC 4-023-03	-

Table 6.3. Comparison of allowable local collapse areas in different documents

BS	Eurocodes	GSA	UFC 4-023-03
15% of floor or roof area or 70 m^2 (whichever is less). Extent of initial damage and one adjacent level either above or below.	The limit of admissible local failure may be different for each type of building. The recommended value is 15% of the floor or 100 m^2 (whichever is smaller) in each of two adjacent storeys.	Structural bay associated with the removed element. 167 m^2 at the floor directly above a removed exterior column or 334 m^2 at the floor directly above a removed interior column.	Removal of exterior column: 70 m^2 or 15% of the total floor area (whichever is less), Removal of interior column: 140 m^2 or 30% of the total floor area (whichever is less)

For some time, the national codes and standards have included general provisions on how to design structure for progressive collapse resistance. However, they still lack more detailed guidance for engineers, since some aspects are not explained in depth. It seems that more detailed commentary with calculated examples would be required. For example, in Eurocode, there is no precise specification what dynamic amplification factor should be used when an equivalent static analysis is employed. This gap is partly fulfilled in U.S. guidelines where more detailed step by step procedures for progressive collapse design are given and the dynamic amplification factor equal to 2, for quasi-static analysis is provided. A few papers (e.g. Powell [58], Ruth et al. [61]) state that this value is too conservative.

The other problem is the not so clear behaviour of buildings when internal or external explosions caused by explosives occur. To shed more light on this issue, appropriate tools should be developed to simulate such phenomena, followed, if possible, by validating experiments.

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A. Formulas for plastic hinges in beams and columns

Default plastic hinges properties of SAP 2000 are based on ATC-40 [6] and FEMA [37] criteria using the following formulas for defining the rotation at yield θ_y:

a) for beams:

$$\theta_y = \frac{ZF_{ye}l_b}{6EI_b} \tag{A.1}$$

b) for columns:

$$\theta_y = \frac{ZF_{ye}l_c}{6EI_c}(1 - \frac{P}{P_{ye}}) \tag{A.2}$$

These refer to the plastic moment capacity as follows

a) for beams:

$$Q_{CE} = M_{CE} = ZF_{ye} \tag{A.3}$$

b) for columns:

$$Q_{CE} = M_{CE} = 1.18ZF_{ye}(1 - \frac{P}{P_{ye}}) \leq ZF_{ye} \tag{A.4}$$

where:
E – modulus of elasticity,
F_{ye} – expected yield strength of the material,
I – moment of inertia,
l_b – beam length,
l_c – column length,
M_{CE} – expected moment strength,
P – axial force in the member,
P_{ye} – expected axial yield force of the member ($P_{ye} = A_g F_{ye}$),
Q – generalised component load,
Q_{CE} – generalised component expected strength,
θ – chord rotation (see Fig. A.1),
θ_y – yield rotation,
Z – plastic section modulus,

Fig. A.1 defines chord rotation for beams. So the chord rotation can be estimated by adding the yield rotation θ_y to the plastic rotation. Alternatively, the chord rotation can be taken as equal to the storey drift.

a) cantilever example   b) frame example

$\theta=\Delta/L$
$\theta_y=\Delta_y/L$

$\theta=\Delta/L$
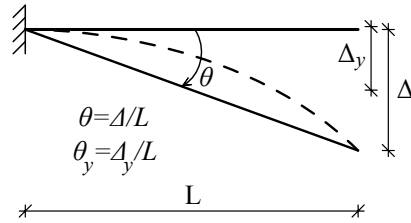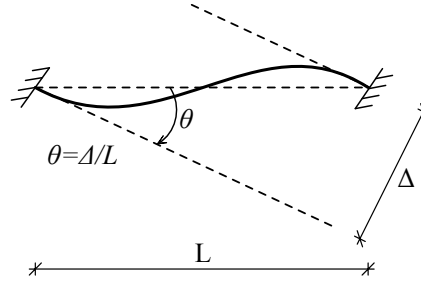
Figure A.1. Definition of chord rotation (Δ - displacement, Δ_y - displacement at yield, θ – chord rotation, θ_y – yield rotation.)

# B. Progressive Collapse and Blast Simulation Techniques

Buildings and other critical infrastructures become often the target of terrorist bombing attacks. A typical pressure wave curve [b1], which will eventually load a structure, at some distance from an explosion is shown in Figure B.1. Its main characteristics concerning damaging effects on structures are the magnitude of the overpressure, the duration of the positive phase and especially its impulse, i.e., the area under the curve over the positive phase. This impulsive load will be delivered to a structure in a few milliseconds forcing it to respond or fail in a peculiar mode.
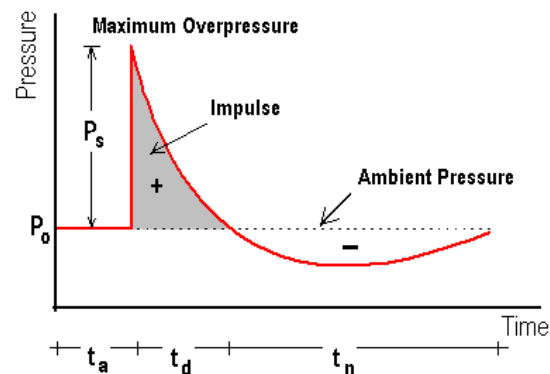
Figure B.1. Blast wave pressure curve characteristics; P_s= maximum overpressure, P_o= ambient pressure, t_a= arrival time, t_d= positive phase duration, t_n= negative phase duration.

A plausible and very likely scenario of failure includes the blasting-off of one or two columns of the building, as schematic shown in Figure B.2, which will trigger excessive deformations and further damage. If not arrested, this may lead to progressive collapse, i.e., the local failure propagates in a disproportionate manner to cause a partial or even global failure. As mentioned above in the report, this was the case of the Government Building collapse at the Oklahoma city bombing.

It is important that the mechanical structure itself mitigates some effects of the explosion and certainly escapes progressive collapse, and this necessitates proper design. Several models, numerical simulations and techniques exist today, which can aid and provide the basis for a blast resistant design of structures. A fundamental prerequisite for the use of these tools is that they first must have been thoroughly validated with reliable experimental data from field tests. However, field tests with actual explosions are expensive and are usually performed within military grounds.
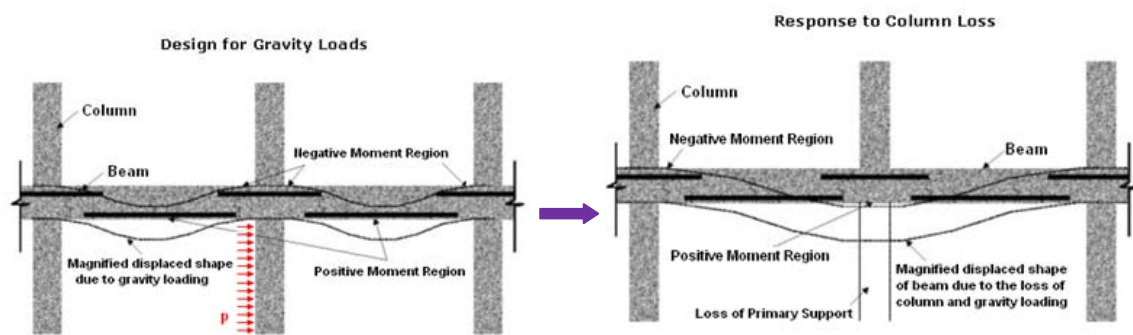
Figure B.2. Schematic loss of the middle column in a reinforced-concrete building frame due to blast pressure p.

Thus alternative testing methods are desirable, and this has been the case at the University of California in San Diego (UCSD), where the first blast simulator facility was built in 2006. As claimed, the effects of bomb are generated without the use of explosive materials. The facility produces repeatable, controlled blast load simulations on full-scale columns and other structural components. The simulator recreates the speed and force of explosive shock waves through servo-controlled hydraulic actuators with special masses attached at their ends, that punch properly the test specimens. The impulse delivered resembles that of Figure B.1 of a real explosion. Such an experiment of the simulation of the "column removal" scenario, reproduced from references [b2,b3], is shown in Figure B.3.
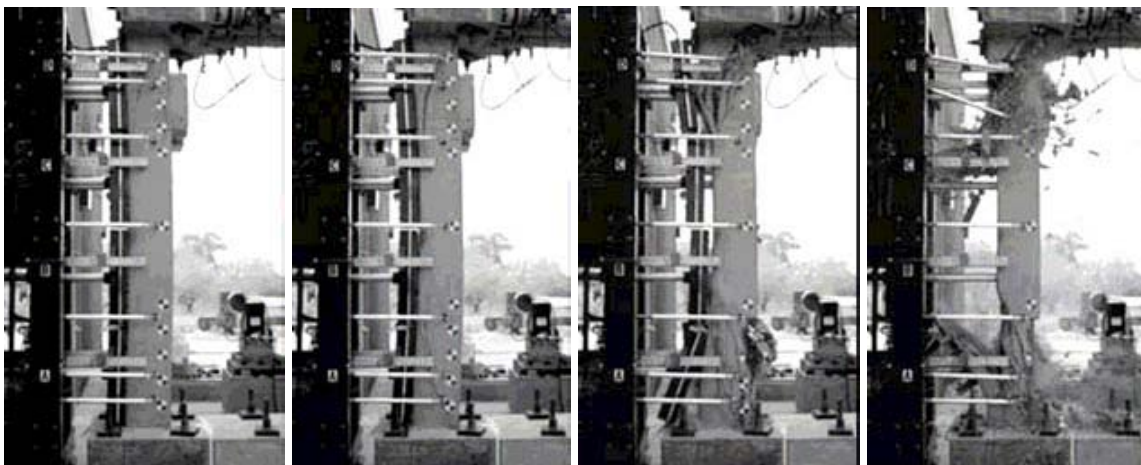
Figure B.3. Photo sequence of the evolution of damage in reinforced-concrete column under blast simulator impact at the UCSD facility.

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Abstract

A technical literature survey has been conducted concerning the problems of building robustness and progressive collapse. These issues gained special interest in construction after the partial collapse of the Ronan Point apartment building in London in 1968. Enhanced interest appeared again after the disproportionate collapse of the A.P. Murrah Federal Building in Oklahoma City in 1995, and the total collapse of the World Trade Center towers in 2001, both caused by terrorist attacks. This report, which is an updated version of the 2009 one, aims at summarising the state-of-the-art in the subject of progressive collapse risk of civil engineering structures. First, a list of main terms and definitions related to progressive collapse are presented. Then, a review of procedures and strategies for progressive collapse avoidance is provided, based on selected EU and US design codes, standards and guidelines. A review of research efforts and results in the field follows, as reported in international journals and conference papers. Different proposals of robustness measures of structures are also examined, and some characteristic cases of progressive collapses of real buildings are presented.

As the Commission's in-house science service, the Joint Research Centre's mission is to provide EU policies with independent, evidence-based scientific and technical support throughout the whole policy cycle.

Working in close cooperation with policy Directorates-General, the JRC addresses key societal challenges while stimulating innovation through developing new standards, methods and tools, and sharing and transferring its know-how to the Member States and international community.

Key policy areas include: environment and climate change; energy and transport; agriculture and food security; health and consumer protection; information society and digital agenda; safety and security including nuclear; all supported through a cross-cutting and multi-disciplinary approach.

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