Design of precast concrete structures with regard to accidental loading

Ir. Arnold Van Acker – Belgium
Former chairman *fib* Commission on Prefabrication

1. INTRODUCTION

A structure is normally designed to respond properly, without damage, under normal load conditions. However, local and/or global damages cannot be avoided under the effect of an unexpected, but moderate degree of accidental overload. Usually, properly designed and constructed structures possesses a reasonable probability not to collapse catastrophically under such loads, depending on different factors:

- the type of loading (internal causes such as gas explosions – external causes such as impact by cars, etc.)
- the degree and the location of accidental loading in regard to the structure and its structural members
- the type of structural systems (skeletal, portal, wall framed structures), and the construction technology (insitu monolithic, precast, mixed precast/steel structures), spans between structural vertical members, etc.

Nevertheless, no structure can be expected to be totally resistant to actions arising from an unexpected extreme cause, but it should not be damaged to an extent that is disproportionate to the original cause, as shown in Figure 1, in which a single slab at one floor level failed during construction.

Fig. 1: Progressive collapse of part of 26 storey apartment building “Ronan Point” (1968)

The normal design procedure to cope with accidental loads consists to admit the collapse of a limited local area of the framework, but to assure that the adjacent areas of the structure surrounding the damage provide for an alternative load pad, possibly in a distorted condition but without leading to collapse of the whole structure.

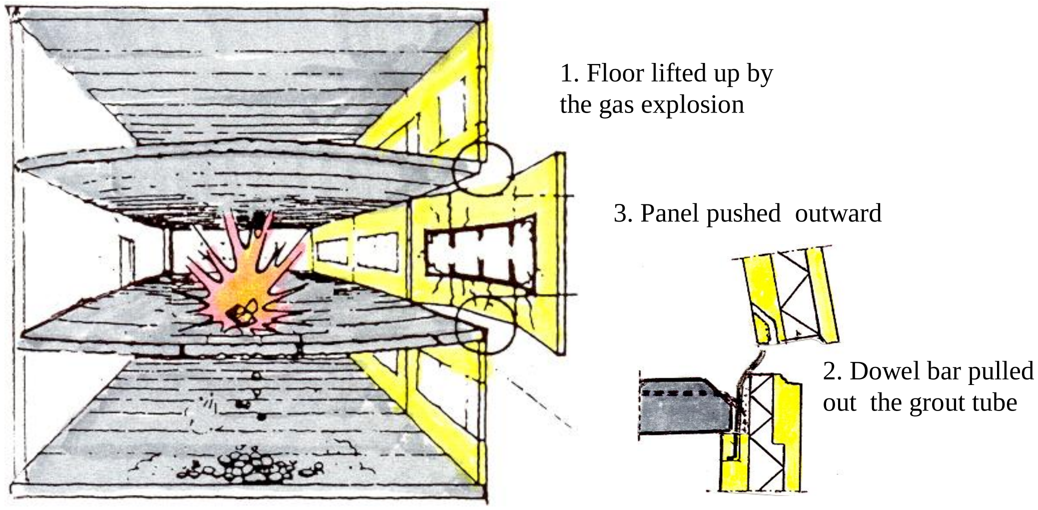

Fig. 2: Scenario of progressive collapse
after blowing-out of a load bearing façade panel due to gas explosion

Progressive collapse is a relatively rare event, as it requires both an accidental action to initiate the local damage and a structure that lacks adequate continuity, ductility, and redundancy to resist the spread of damage. It is technically very difficult and economically prohibitive to design buildings for absolute safety. However it is possible to construct buildings that afford an acceptable degree of safety with regard to accidental actions.

The *fib* Commission on Prefabrication has studied the present knowledge on the subject and drafted a Guide to good practice for the design of precast structures against progressive collapse. The aim of this paper is to give an overview of the document which will be published in the coming months.

2. STRATEGIES TO COPE WITH ACCIDENTAL ACTIONS

The basic physical protection strategies for buildings to cope with accidental actions are given hereafter. They are not alternatives, but can be used in combination in the same building independently of the construction method used, to reduce the risk of progressive collapse. Before starting the analysis, it is always recommended to examine the possible risks of a project, and the severity of the consequences. The following items are dealt with:

1) Categorisation of buildings
2) Systematic risk assessment
3) Reduce the risk of accidental actions;
4) Conceptual measures to prevent the effect of accidental actions.
    4.1 Architectural design
    4.2 Structural design
        a. Tie force approach (indirect design approach)
        b. Alternative load path approach (direct design approach)
        c. Specific load approach (direct design approach)

2.1 Categorisation of buildings

The American and European standards prescribe a minimum level of protection of building structures against accidental actions in function of possible consequences, primarily depending on the extent (more specifically the height) and the function of a building. Usually, buildings are classified in so-called consequences classes:

- Consequences class 1: low (limited consequences)
- Consequences class 2a and 2b: medium
- Consequences class 3: high

Eurocode EN 1991-1-7 provides a table that gives recommended consequences classes for different types and occupancies of buildings. The following Table combines these recommendations with recommended design strategies from Eurocode EN 1991-1-7. The type of approach and the recommended level of resistance to accidental actions are based on the potential consequences of the event.

Consequen-ces class	**Type of building and occupancy**	**Design strategies (not from Eurocode)**
Class 1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or areas where people do go, than a distance of 1½ times the building height	Provided a building has been designed in accordance with the rules given in national or international standards for satisfying stability in normal use, no further specific consideration is necessary with regard to accidental actions from unidentified causes.
Class 2a Low Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1 000 m² floor area in each storey. Single storey educational buildings. All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 m² at each storey.	Buildings should be designed in accordance with the requirements of *the indirect approach.* Effective peripheral and internal ties should be provided according to Section 7.1.3 respectively for framed and load-bearing wall construction. Vertical ties are not strictly required but always recommended.
Class 2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m² but not exceeding 5000 m² at each storey. Car parking not exceeding 6 storeys	Horizontal and vertical ties should be provided according to the provisions set fort in Section 7.1.3.In addition, the building should be designed in accordance with the requirements of the *alternate path approach.* The building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall as defined in Section 7.2, the building remains stable and that any local damage does not exceed a certain limit. Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, then such elements should be designed in accordance with the *specific load resistance method.* (see Section 7.3). In the case of wall frame buildings, the notional removal of sections of wall, one at a time, is likely to be the most practical strategy to adopt.

Class 3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators. Buildings deemed to be high-risk targets. Buildings containing hazardous substances and /or processes.	Distinction is made between buildings with normal occupancy and buildings with a high occupancy or significant consequence of an accidental action. 1) Buildings exceeding the limits of Class 2a and 2b. This category of building should be either a) designed in accordance with the requirements of the *alternative path approach* as specified in Section 7.2 or, b) a systematic qualitative risk assessment of the building should be performed and the required improvements based on this assessment implemented. 2) Buildings with high occupancy and Stadia with a capacity of more than 5000 persons. For this category of building the consequences of accidental actions can be significant and a systematic risk assessment of the building should be undertaken and the required improvements based on this assessment implemented. 3) Buildings deemed to be high risk or buildings containing dangerous substances, or where dangerous processes are carried out. For this category of building a systematic risk assessment of the building should be undertaken and the required improvements based on this assessment implemented.

Note 1: For buildings of more than one type of use the "consequences class" should be that relating to the most onerous type.
Note 2: In determining the number of storeys, basement storeys may be excluded provided such basement storeys fulfil the requirements of "Consequences Class 2b Upper Risk Group".
Note 3: Consequences class for building types not specifically covered should be taken as the closest similar type.

Table 1. Categorisation of consequences classes and design strategies.

2.2 Systematic risk assessment

The purpose of a systematic risk assessment of a building project, is to detect and denominate the potential risk of occurrence of accidental actions, and their related effects. It is a decision support in an early stage of the design. The results may help to choose a design strategy to minimize the risk of progressive collapse.

There are two types of systematic risk assessment:
a) a quantitative assessment, whereby the effects of an accident (collapse) are quantified and chances of an occurrence of the risk scenario estimated.
b) a qualitative risk assessment, where the search for the weak spots takes place without the quantitative weighing. The argumentation for omitting this weighing is the arbitrary character of attributing the said gradations.

2.3 Measures to reduce the potential for progressive collapse

Initial local damage can result from intentional explosions, accidental explosions, vehicle impacts, fire, or other abnormal load events. There are several methods to reduce the potential for accidental actions and/or to reduce the effect thereof. Generally these methods can be divided into four categories: elimination of the initial cause, site conditions, architectural concept and structural systems.

a) Elimination of the initial cause
Accidental actions could be avoided by eliminating the initial cause. However, in most cases it is not realistically possible to completely exclude the occurrence of all possible accidental actions. One of the main reasons for that is the fact that we are not able to identify all possible accidental actions that may occur during the life time of a building. The risk of explosion of domestic gas could be avoided by prohibiting gas installations in buildings.

b) Site conditions and measures for protection
The placement of the building on the site can have a major impact on its vulnerability. Barriers along the secured perimeter of a building should have anti-ram capability consistent with the size of the vehicles and the maximum achievable velocity. Buildings abutting on watercourses may be protected against ship collision by protective bollards, etc. Landscaping features that create an obstacle course may also be used to prevent a vehicle from ramming into the building.

c) Architectural concept
The shape and lay-out of the building can have a contributing effect on the overall damage to the structure. For new buildings, a regular, uniform layout of structural elements (beams, columns, and walls) can have a significant impact on the ability of the structure to withstand progressive collapse. Regularity in design allows for continuity of strength, greater redundancy, and hence capacity for redistribution of load should an element fail due to impact or accident. Irregularities, such as reentrant corners and overhangs, are likely to trap the shock wave of an explosion, which may amplify the effect of the air-blast.

d) Reduction of the action effect
Sandwich cross-sections with hard and soft layers will enable a dissipation of energy. The pressure could also be reduced by security walls. Another solution is to design the window fixings in such a way that they could be used as venting panels. It is recommended that the risk of injury to persons from glass fragments or other structural members be considered.

3. DESIGN METHODS TO PREVENT PROGRESSIVE COLLAPSE

There are three design methods, which should not be considered as different alternatives, but it is recommended that the several approaches are used in combination. The alternatives are:

a) indirect design method;
b) alternative load path method;
c) specific load method;

The alternatives mentioned under b) c) are also defined as direct design approaches. Each of the design methods is based on assumptions and conditions that offer different technical advantages and disadvantages. Alternative a) is always needed to cover unpredicted events; the two other are always based on certain assumptions.

3.1 Indirect design method

The *Indirect Design method* is also called “Tie Force Approach”. Resistance to progressive collapse is considered indirectly through provision of minimum levels of strength, continuity and ductility through the whole structure by application of ties. The dimensions of these ties are mostly based on “deemed to satisfy rules”, or other comparable, more or less arbitrary assumptions. Adopting this method should provide a building with sufficient robustness to survive a reasonable range of undefined accidental actions.

Ties are continuous tensile elements consisting of reinforcement bars or tendons, placed in cast in-situ infill strips, sleeves or joints between precast elements, in longitudinal, transversal and vertical directions. Their role is not only to transfer normal forces between units, originating from wind and other loading, but also to give additional strength and safety to the structure to withstand, to a certain extent; loading conditions termed as accidental actions: settlements, gas explosions, vehicle or aircraft collision, tornado’s, explosive bombings, etc.

The building is mechanically tied together to enhance continuity and ductility so that alternate load paths can be developed. The types of ties that must be provided depend on the type of structure. They include both horizontal and vertical ties with particular emphasis on tying the perimeter of the structure to the internal core. The tying system must be effectively continuous around or across a building. Along a particular load path, different structural elements may be used to provide the required tensile capacity, providing that they are adequately connected. Figure 3 illustrates the ties required for skeletal or bearing wall structures.

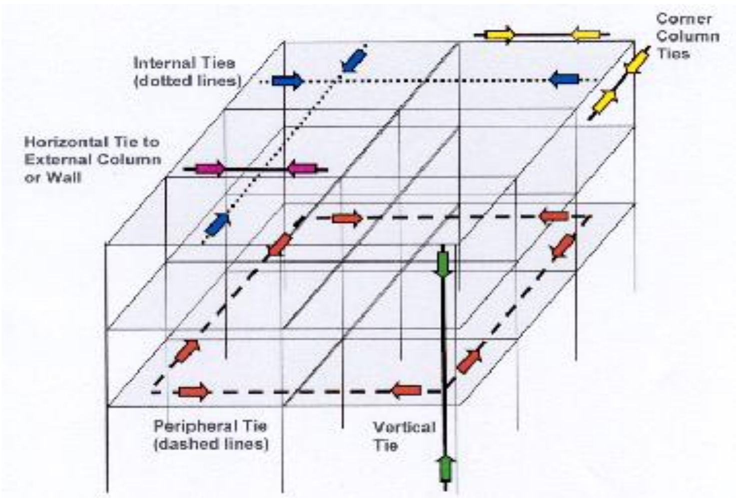

Fig. 3: Schematic of tie forces in a skeletal or bearing wall structure

Requirements set forth by Eurocode 1Part 1-7

a) Skeletal structures

- Internal ties

$T_i = 0.8(g_k + \psi q_k).s.\ell$ or 75 kN, whichever is the greater (1)

- Peripheral ties

$T_p = 0.4(g_k + \psi q_k).s.\ \ell$ or 75 kN, whichever is the greater (2)

- Vertical ties
  - · Each column or wall should be tied continuously from the foundations to the roof level
  - · The columns and walls should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey.

s is the spacing of the ties

ℓ is the span of the tie (in the not damaged structure)

ψ is the relevant factor in the expression for combination of action effects for the accidental design situation (see Section 4.2.2)

b) Load bearing wall structures

- Internal ties

  T_i = the greater of $T_i = F_t$ or

$$T_i = F_t \frac{g_k + \psi q_k)}{7.5} \cdot \frac{z}{5} \qquad (3)$$

- Peripheral ties

  $T_p = F_t$

- Vertical ties
  - · Each wall should be tied continuously from the foundations to the roof level
  - · The vertical ties may be considered effective if:
    - a) the clear height of the wall, H, measured in metres between the faces of floors or roof does not exceed 20t, where t is the thickness of the wall in metres,
    - b) if they are designed to sustain the following vertical tie force T:

$$T = \frac{34A}{8000}\left(\frac{H}{t}\right)^2 N \text{ or } 100 \text{ kN/m wall whichever is the greater} \qquad (4)$$

    - c) 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. Alternative load path method

The *alternative load path method* presumes that a critical element is removed from the structure as a result of an accidental loading. The structure is required to redistribute all relevant loads in the design with regard to progressive collapse, to the remaining undamaged structural elements. The connections and tie-reinforcement should be designed to resist the resulting actions. The success of this approach depends on the correct selection of the critical elements.

The alternative load path method implies that:

- the primary local damage must be bridged by an alternative load-bearing system. The transition to this system is associated with dynamic effects that should be considered
- the structure in it whole must be shown to be stable with the local damage under the relevant load combination

3.2.1 Primary local damage

The practical analysis procedure consists to notionally remove an external or internal load bearing unit, one by one, at critical locations. For façade columns, those locations are for example near the middle of the short side, near the middle of the long side, and at the corner of the building. Internal columns or wall panels must also be removed at other critical locations. Elements should also be removed at locations where the plan geometry of the structure changes significantly. For each plan location, the alternative load path analysis is only performed for the element on the ground floor or parking area floor and not for all stories in the structure.

3.2.2 Actions to consider in the design

The actions to be applied in the design of an alternative load path are normally the self-weight of the structure and the frequent or quasi permanent values of the design life load and snow load. The wind load may be neglected. The choice of the frequent or quasi-permanent value for the variable actions depends on the probability of simultaneous occurrence of the different actions. In case of earthquake design, the quasi permanent values may be used, since an earthquake is a random event in time, unlikely to coincide with live-load peaks. Explosions with domestic gas are also a random event in time, but a terrorist attack may not be so random. It is the responsibility of the designer to choose the adequate combination factors, depending on the potential risk of occurrence and the related effects, for instance the type of building, the activities envisaged to take place in the building, as well as the number of people expected to occupy the building.

The fundamental combination formula of actions to take into account is given hereafter.

$$\omega[G \cdot \gamma_G + (\psi_1 or \psi_2) \cdot Q_{LL} + \psi_2 Q_{SN} \cdot \gamma_{Q_2} \qquad (5)$$

where ω is an amplification factor
G is the self-weight of the structures
γ_G = 1,0 (accidental situation)
ψ_1 see table A1.1 EN 1990
Q_{LL} life load
γ_Q = 1,0
ψ_2 see table A1.1 EN 1990
Q_{SN} snow load
γ_Q = 1,0

Practical values for load factors for buildings can be found in Eurocode 1.

For linear and non-linear static analyses of all construction types, the amplification factor $\omega = 2.0$ should be applied to those bays immediately adjacent to the removed element and at all floors above the removed element. For the rest of the structure, the amplification factor $\omega = 1.0$. For non-linear dynamic analyses of all construction types the amplification factor $\omega = 1.0$

Upward loads on floors and slabs

Since the pressure from an explosion is omni-directional, measures should be taken to prevent floors above the explosion, to lift up and break. To this end, in each bay and at all floors and the roof, the slab/floor system must be able to withstand a net upward load of the following magnitude:

$$F = 1.0\ G_k + 0.5\ Q_k \qquad (6)$$

3.2.3 Mechanisms to provide for alternative load paths

The following mechanisms can be used to provide for an alternative load path in multi-story precast concrete structures.

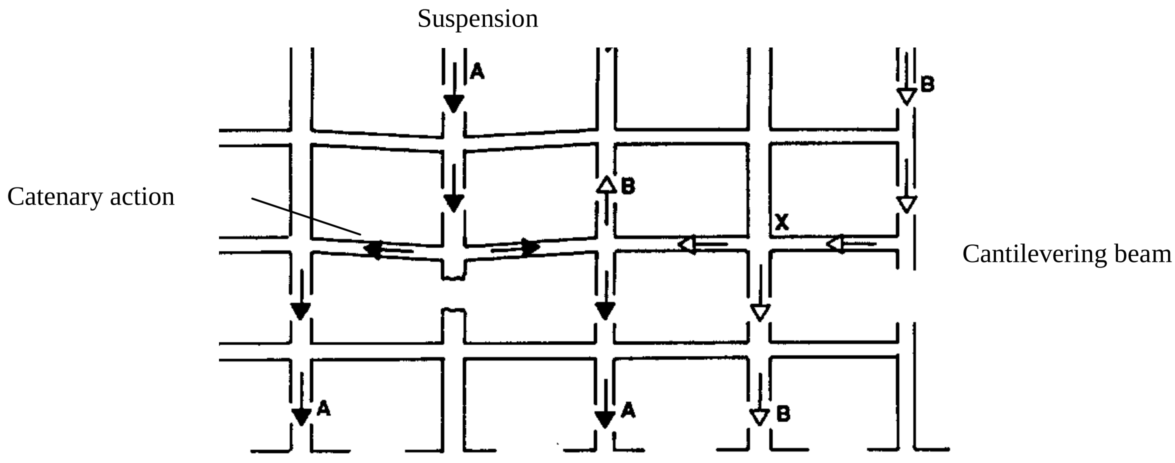

Fig. 6: Alternative means of protection against progressive collapse in skeletal structures

a) Suspension of the elements to the intact upper structure above the damaged area. This is realized by vertical ties from foundation to roof level in all columns and walls.

b) Cantilever action of the surrounding structure, for example in case of failure of a corner column or wall panel. The horizontal tie reinforcement on top of the floor beam or wall panel will take up the tensile stresses of the cantilever. To this effect the tie-reinforcement should be duly connected to the beam or panel, for example inside projecting hairpins at the top of the units.

c) Bridging of the damaged area by catenary action of the tie beams. In the event of accidental damage to a column, it can no longer carry any of the force and the ultimate design load must be distributed to other members, to avoid total failure. The loss of support means that the beam has effectively doubled in length and the excess forces in the system must be carried through catenary action. When the beam deflects, the tie bar is strained and a tensile force is mobilised. The excess loads are transmitted through the tie bar via the links and with increased deformation, a new equilibrium state will develop.

d) Prevention of damaged floors from falling down on the underlying structure. Progressive collapse is often the result of accumulation of debris from successive collapsing floors falling on the lower floors. The longitudinal ties anchoring the floor slabs to the support structure are best placed in the middle of the floor depth to allow for maximum efficiency and deformability.

3.2.4 Practical analysis

In the following, the application of the above mechanisms are discussed specifically for precast frame structure buildings in comparison to a cast in-situ one.

When a multi-storey concrete frame building is subjected to a sudden column loss along the perimeter, the ensuing structural response is dynamic, leading to large deformations in the floor structure.

In a monolithic cast in-situ building, the floor including the edge beams will act as a whole in the redistribution of the load to the surrounding structure. The edge beam has continuous bottom and top reinforcement resulting in a cable shape deformation, and the connection between the slab and the edge beam will be sufficient to follow the deformation of the edge structure and hence to contribute to the load transfer.

In a precast structure, the floor and beam interaction in the transfer of the accidental loading will depend on the structural lay-out and the connections. Contrary to a monolithic structure, the deformations will mainly concentrate in the connections between the beams and floors, provided that the remaining columns can take the redistributed gravity load. The precast floor beams, supported previously by the removed column, will normally remain completely rigid and keep their original shape.

The tie-reinforcement above the beam will function as a catenary, and take up the beam support reaction at the removed column, via the projecting stirrups in the beams (Figure 4). Indeed, on the assumption that the prestressed beam will remain perfectly rigid under displacement, the suspension force of the floor beams to the catenary tie reinforcement will concentrate at the beam ends. The actual scenario is of course very much influenced by the detailing.

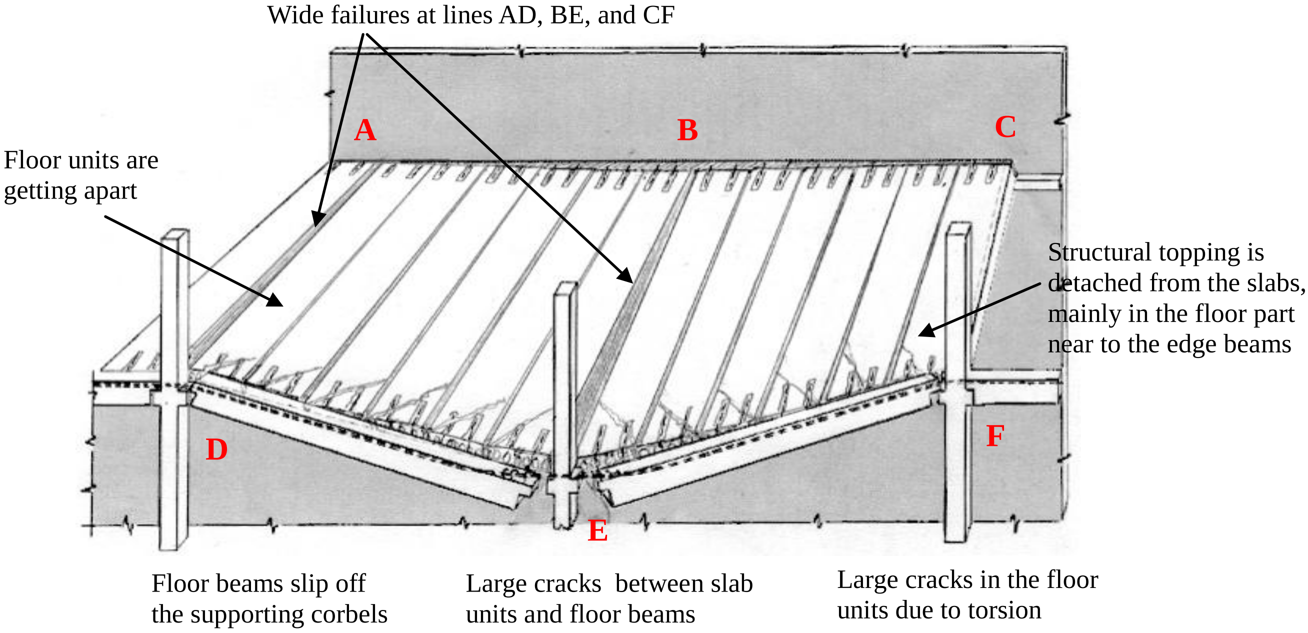

Fig. 4: Possible scenario of the structural behaviour of a precast frame structure after sudden column loss due to accidental actions

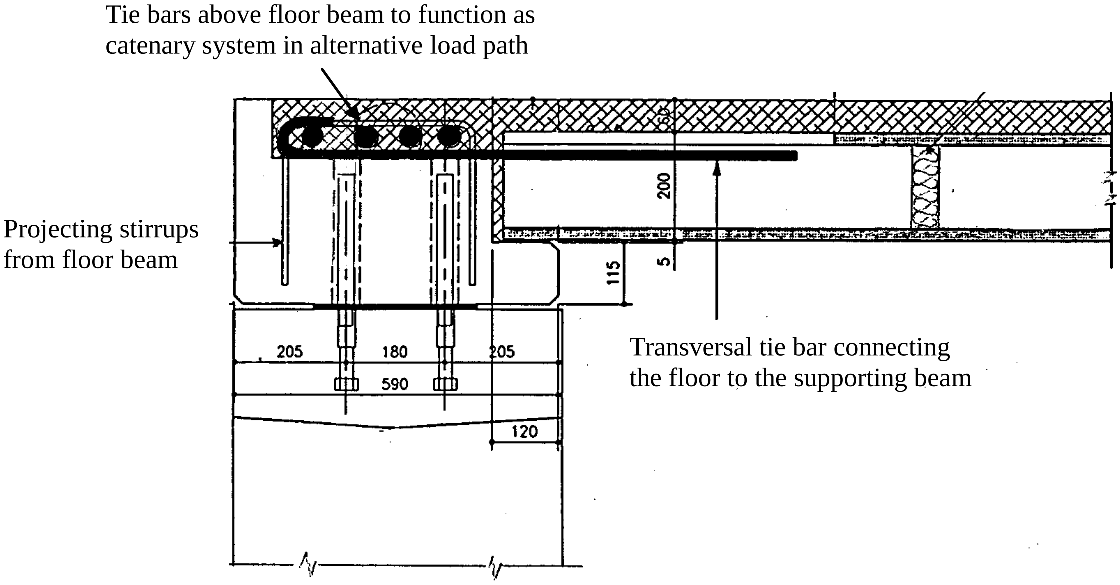

Fig. 5: Example of connection between floor beam and peripheral tie reinforcement

In the model illustrated by Figure 4, the primary load transfer to the surrounding structure is assured by the floor beams alone, without major intervention of the transversal tie reinforcement in the structural floor topping. The motivation is given hereafter.

- The precast beams and floor units are in prestressed concrete, and their rigidity is much larger than the rigidity of their connections. Hence, the elements under displacement can be assumed to be perfectly rigid.
- The floor beams are connected to the columns by means of dowel bars in the corbels and tie bars on top of the beams. Depending on the shape of the beam ending (straight ending or half joint) and the size of the dowel bars, the beams will remain connected to the corbel, or slip off from it. In Figure 4, the beams are connected to the corbel at half height, and the risk of getting off from the corbel is larger than in case of a straight ending beam, where the

dowel bar goes through the full height of the beam.

- The length of the deformed floor support D-E-F in Figure 4 is much greater than the length of the opposite support A-B-C. As a consequence, the slabs will split apart, and be subjected to torsion due to the non-planarity of their supports. The longitudinal joints at the lines AD, BE and CF will probably show larger openings than the other joints. In case of hollow core floors, the connecting reinforcement between floor units and beams should be positioned in open sleeves in the units and not in the longitudinal floor joints since these will open during the deflection.
- The length of the deformed structure at the axis BE is much greater than at the axis AD and CF, at least when it is assumed that the column at E will not move inwards. This could happen when all the upper floors deform in the same way. Depending on the position of the longitudinal tie bars in the cores (top, middle or bottom), there might be a serious risk that the floors will slip off the booth of the supporting beam and hang on the connecting bars with the beams.
- The structural topping on the floor will most probably be detached from the floor units in the vicinity of the support line D-E-F during the collapse, unless the reinforcement in the topping is effectively connected with the HC's through projecting links cast in filled sleeves. Connecting links anchored in the longitudinal joints between the floor units will not work because of the splitting apart of the units.
- As a consequence, in the most unfavourable scenario, the full gravity load will be taken by the peripheral tie bars anchored to the floor beams, and the deformations in the transition to a catenary system will concentrate in the joints between these beams and the supporting columns.

3.3 Specific load approach

The *specific load approach* requires all critical load-bearing members (key elements) to be designed and detailed to be resistant to a specified design value of accidental load. Selection of the appropriate type and magnitude of the accidental load is critical and will vary with the occupancy and type of building. Because of the subjective nature of this approach the other parts of the structure, such as tying the offending component into the structure, should be designed according to the alternative load path method.

A key element should be capable of sustaining an accidental design action of A_d applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the ultimate strength of such components and their connections. Such accidental design loading should be applied in accordance with equation (6.11b) in Eurocode EN 1990 and may be a concentrated or distributed load. According to EN 1991-1-7, the recommended value for A_d for building structures is 34 kN/m².