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STRUCTURAL ROBUSTNESS OF MONOLITHIC AND PRECAST RC BUILDING

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Research area: *theory of RC structures and concrete structures*

ABSTRACT

During the long service life a structure could be exposed to some incidental (abnormal) actions which are not obtained in conventional design process. These events are unpredictable and it is not feasible and economical to include them into design considerations. Many cases of progressive structural fracture occurred in the past (the well-known case of Roman Point, 1968) and others have been the consequence of a lack of robustness properties. The definition is confined to the ability of structures to withstand an abnormal event, involving a localized failure with limited levels of consequences, or simply structural damages. Robustness-resistance to progressive collapses of buildings is the characteristic which defines the structure strength in terms of integrity and redundancy. Progressive collapse is defined as situation where local failure of primary structural components leads to the collapse of adjoining members, which leads to additional chain collapse. This property is strongly related to the internal structural characteristics (redundancy, ductility and joints), as well as the scenario of events. The paper presents the importance of robustness and the collapse-resistant structure for reducing the progressive collapse of RC monolithic and precast buildings. The various ways of achieving the robustness, integrity and ductility of the structure are discussed. Review of literature and some code provisions and recommendations, such as COST TU0601, EN, USA, Canada and UK, for designing RC building structures for progressive collapse performance are given, too.

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1. Introduction

Robustness of structural systems has gained renewed interest following the collapse of the World Trade Centre towers on September 11, 2001 [5]. Robustness of structural systems has lately been intensively used, and slowly introduced into the engineering practice, especially after the analysis of their consequences. The reinforced concrete structures (RC), especially precast concrete structure of buildings, are vulnerable to various adverse impacts. The overall system safety depends on the localisation of damage, structural topology, materials used and level of stresses in some parts of the structure. According to [2] accidental design situation refers to conditions applicable to the structure (local overloading) or to exposure impact or the consequences of localised failure. Robustness-resistance to progressive collapses of buildings is the characteristic which defines the structure strength in terms of integrity and redundancy. When any element fails, the remaining elements of the structure seek alternative load path to redistribute the load applied to it. As a result, other elements may fail causing failure mechanism [9]. The probability of a progressive collapse must be limited to a generally accepted value [6].

The term “progressive collapse” was first used in the UK following the partial collapse of a precast concrete wall at Roman Point, London in May 16, 1968 (**Fig. 1a**). A gas explosion in a corner room in the eighteenth floor blew out one of the external walls, and because of inadequate structural continuity between the wall and floor elements, the removal of one wall element was sufficient to cause the total collapse of a part of floor area per storey over the height of the entire building [14] and [23]. This building structure was of a large panel concrete building. Generally, precast concrete structure is more inferior to structures of monolithic concrete buildings. Reason for that lay in sensitivity of joints and connections and orientation of precast floor planks [12]. The collapse following the terrorist attacks on the Murrah Federal Office Building in 1995 is shown in **Figure 1b**.

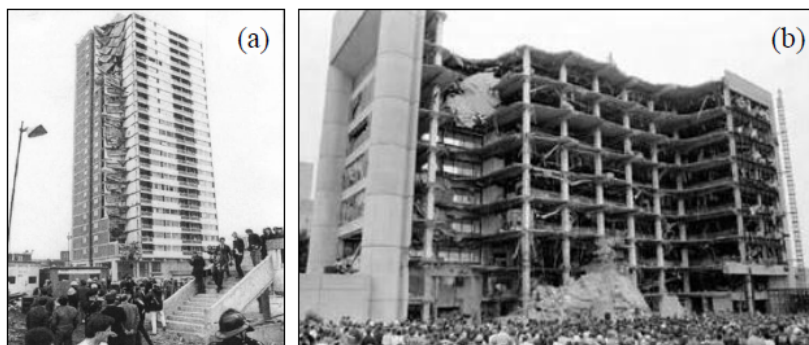


Figure 1. Examples of progressive collapse
a) Roman Point Apartment Tower; b) Murrah Federal Office Building, after [46]

The definition of robustness used in EN1991-1-7:2006 [15] is the following: “Robustness is 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”, attention is predominantly directed towards buildings.

The methods for defining and evaluation of robustness are currently being improved since this phenomenon is accepted and insufficiently studied in the world. A considerable improvement in defining robustness was made in the report: COST1 Action TU-06012-Robustness of Structures [9]. Many of the contemporary building codes specify that the

consequences of structural failure should not be disproportional to the effect causing the failure. The level of robustness of a structure should be analysed in terms of the causes and consequences of failure; i.e. the consequences of structural damages should not be disproportional to the original cause. The EN 1990 [15], EN 1991-1-7 [16], EN 1992 [17] provide the general principles for achieving structural robustness in the case of fire, explosion, impact, and the consequences of human errors, impact and gas explosion.

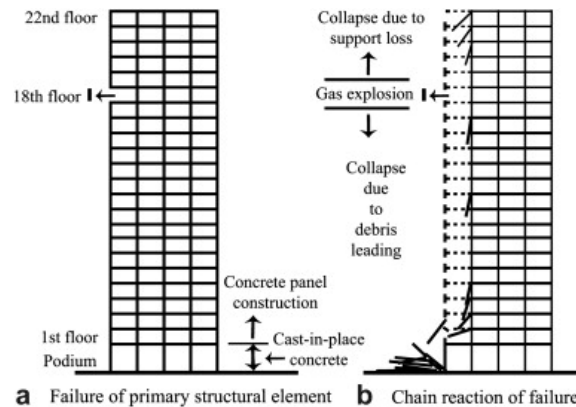


Figure 2. Schematic show of Roman Point collapse

This paper attempts to emphasise the impact of accidental (exceptional) action on reliability and robustness of RC and Precast concrete structures (PCS) of buildings, which are the most common type in this region. Review of papers and Codes related to various methods to reduce risk from accidental loads is given. This paper provides an overview of important and applicable methods of robustness evaluation from the topical references, as well as the importance of redistribution of impacts of structural elements after the undesirable consequences. The paper stresses on precast concrete structures which are more sensitive under accidental (abnormal) actions than classical RC structures.

2. Assessment of structural vulnerability and robustness

When a structure is exposed to an event, some damage may occur. Collapse resistance is a property that is influenced by both structural features as well as possible causes of initial failure. The 'risk' concepts provide a rational and comprehensive way of making robustness related decisions. *Vulnerability* describes the sensitivity of a structure to damage events. The structure is vulnerable if small damage lead to disproportionate consequences. For robustness quantification, based on a probability, an event tree [1] and [27] is often used. For each of these states, there is a probability that a system failure (F) will result. When considering structural failures, consequences are divided into: direct and indirect consequences. Direct consequences are related to the vulnerability of the structures. Indirect consequences are related to the loss of system functionality, as the result of local failure, and are related to the robustness of the structures. A robust system has to keep its integrity even in the case of accidental actions. The relationship between robustness, vulnerability and consequences in terms of the risk assessment framework adopted to analyse the robustness of structures can be presented as:

Exposure → vulnerability → Robustness (COST Action TU0601) [10].

Definitions in EN [16] are derived from ISO 2394, ISO 3898, ISO 8939 and ISO 8402: accidental *action* is usually of short duration but of significant magnitude, and it is unlikely to occur on a given structure during the design working (service) life. An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken. Impact, snow, wind and seismic action may be variable or *accidental* actions, depending on the available information on statistical distributions. *Accidental design situation* involves exceptional conditions of the structure or its exposure, including fire, classes divided as follows [17]: CC1-consequences class 1: Low consequences of failure; CC2-consequences class 2: Medium consequences of failure; CC3-consequences class 3: High consequences of failure.

The consequences of failure vary significantly and depend on the following factors: hazard, property of structures, and location of damaged element and robustness of structures. A progressive collapse is a chain reaction which occurs after the damage or loss of load bearing capacity of a small part of a structure under incidental loads or in case when some bearing elements fail. In a robust structure no damage disproportionate to the initial failure will occur. The quantification of robustness or related characteristics such as vulnerability are presented and discussed in [2], [4], [6], [7], [12 – 15], [23 – 30], [40 – 41] and [43 – 45].

In practice, structures may also have additional requirements of the stakeholders related to the people affected by economic, social, environmental and business consequences. A robust system is considered to be one where indirect risks do not contribute significantly to the total system risk. With this in mind, the following index of robustness (denoted I_{Rob}) is proposed, which measures the fraction of total system risk resulting from direct consequences:

$$I_{Rob} = \frac{R_{Dir_1}}{R_{Dir_1} + R_{Ind_1}}. \quad (1)$$

The index takes values between zero and one depending upon the source of risk. If the system is completely robust and there is no risk due to indirect consequences, then $I_R = 1$. At the other extreme, if all risk is due to indirect consequences, then $I_R = 0$, given in [1]. According to this definition, a structure which is designed to have strong “key elements”, but which may progressively collapse once a key element is removed, is a structure with a low robustness index. An alternative definition associated with the redundancy of a system was also given in the terms of the reliability index using a “redundancy factor” β_R by:

$$\beta_R = \frac{\beta_{int.}}{\beta_{int.} - \beta_{damaged}}. \quad (2)$$

This equation is suggested by Frangopol and Curley (1987) and is given in many papers.

The methods of quantifying robustness help to study the behaviour of structural systems and their sensitivity to various parameters. For sufficiently robust structures, failure consequences can be significantly reduced. The European research project COST Action TU0601 [10] has been initiated to establish better understanding of the aspects related to robustness. Available robustness indices can be divided into three levels with increasing complexity:

1. Deterministic such as the deterministic reserve strength ratio (ISO 19902).
2. Reliability-based such as the redundancy index derived from failure probability of a damaged and intact structural system.
3. Risk-based proposed a definition of a risk-based index – ratio of the direct risk over the total (direct + indirect) risk. Consequences are divided into direct consequences (proportional to the initial damage) and indirect (disproportional) consequences.

The probability of a progressive collapse must be limited to a generally accepted value. Those value $P(F)$ as a result of an abnormal event can be represented as a chain of partial probabilities [10] and [42]:

$$P(F) = P(F|DH) \cdot P(D|H) \cdot P(H), \quad (3)$$

where: $P(H)$ denotes the probability of an abnormal event that threatens the structure or more generally a hazard H for the structure; $P(D|H)$ is the probability of local damage D as a result of the event H ; and $P(F|DH)$ denotes the probability of the failure F of the structure as a result of local damage D by H .

Definitions related to robustness are shown in [17]. *Collapse resistance* is a property that is influenced by both structural features and possible causes of initial failure. The term is defined as insensitivity of a structure to accidental circumstances (low probability events).

Continuity refers to the continuous connection of components as well as continuous reinforcement of concrete components. These properties improve integrity, redundancy and/or local resistance.

Ductility is the ability of the component or structural system to withstand large post-elastic deformations and increases the robustness of structure.

Damage tolerance is compatible with the term robustness and refers to the ability of a structure to resist a continuous local deterioration due to corrosion or similar.

Integrity is related to requirements for continuity, ductility and redundancy. Enhanced integrity may mitigate progressive collapse. *Ties* contribute to the ability of a structure to redistribute or transfer loads along this load path based in large part on the interconnectivity between adjacent members.

Key element is a structural member upon which the stability of the remainder of the structure depends.

The *redundancy* of a structural system is related to its strength and shape, where particularly the degree of static indeterminacy (number of potential alternative load paths) is important. Structural *redundancy* refers to the multiple availability of load-carrying components of multiple load paths which can bear additional loads in the event of failure. Besides, global design strategy based on classical requirements on sufficient ductility and tying of elements can be applied to achieve some degree of redundancy [21]. For conventional structures it should be possible to include all relevant collapse origins in the design.

There are a number of ways to obtain resistance to progressive collapse and in the ASCE 7 – 10 two ways of design, direct and indirect design, are described. The *direct design* considers the resistance to progressive collapse *explicitly* during the design process itself. This can be obtained by the *alternative load path method* which allows local failure to occur without major collapse, because the other load path(s) will allow the damage to be ‘absorbed’. The structural integrity of a structure may be tested by analysis to ascertain whether alternative paths around hypothetically collapsed regions exist. In addition, the Standard recommends the *specific load resistance method*. This method seeks to provide sufficient strength to resist failure from accidents or misuse. This may be provided in regions of high risk since it may be necessary for some elements to have sufficient strength to resist abnormal loads in order for the structure as a whole to develop alternate paths. The design philosophy necessitates that accidental actions are treated in a special manner with respect to load factors and load combinations [26].

3. Design Methods and Analysis of Robustness

As building designers cannot design for every hazard that a building may be subjected to in its lifetime, a general design approach is required to account for the risks associated with low-probability high-consequence events. There are, in general, three alternative approaches to designing structures to resist progressive collapse: Improved interconnection or continuity; Key element design; Notional element removal. These approaches can be classified into terms of indirect and direct design approaches.

During designing the following principles and measures necessary for the design and evaluation of structural robustness should be taken into account: identification and specification of accidental design situations and actions; verification of overall stability and stiffness of a structure; verification of vulnerability of structural elements and details; indirect design of alternative load paths by means of internal and perimeter, horizontal and vertical ties.

Examples of *accidental* and/or *exceptional events* to be considered are loads by a gas explosion, collision loads, impact loads on roofs, loads by extremely high soil water, loads due to fire or earthquakes, consequences of human errors, deterioration effects, etc. To avoid damage or to ensure robust behaviour [17] in clauses 2.1 (5) and (6) the appropriate choice of one or more of the following measures is required:

- Avoiding, eliminating or reducing the hazards which the structure can be subjected to;
- Selecting a structural form which has low sensitivity to hazards considered;
- Selecting a structural form and design that survive adequately the accidental removal of an individual element or a limited part of the structure or the occurrence of acceptable localised damage;
- Avoiding as far as possible a structural system that can collapse without warning;
- Tying the structural members together.

Indirect design methods consist of various prescriptive measures of improving a structure's robustness. These methods can be implemented without the need for any additional analysis. Indirect design methods are incorporated into most major codes and guidelines. The provisions are usually in the form of prescriptive requirements for minimum joint resistance, continuity and tying between the members [17 – 18] and [21 – 22]. Both horizontal and vertical ties should be included. The provision of horizontal ties is based on the concept that, following the loss of a support, the remaining structure will support the loads through catenary action.

In contrast with indirect methods, direct design approaches rely heavily on structural analysis and can benefit significantly from the use of sophisticated analysis techniques, such as nonlinear and/or dynamic analysis. Two commonly applied approaches to reduce the potential for disproportionate collapse are *key element design* and *notional column removal*. In *linear static analysis* (often using ETABS V9.7) dynamic effects can be indirectly considered by assuming an equivalent static load based on a constant amplification factor, typically taken equal to 2.0. In nonlinear static analysis the sudden removal of a structural component results in an immediate change in the structural geometry and both geometric and material nonlinearity. The inclusion of these nonlinearities is required to account for membrane effects, as well as to allow for accurate representation of inelastic response and P- Δ effects. Linear dynamic analysis is easier for application than nonlinear dynamic analysis, but complex for practice. It is one of the reasons why in practice it is resorted to modelling the structures using complex software, based on 3D modelling.

The most rigorous approach for applying the notional column removal method is through the use of nonlinear dynamic analysis. This method dynamically removes a member from the structure, which is then analysed through taking account of both the geometric and material nonlinearities and is the impact of failed members on other portions of the remaining structure.

The **key element** design approach increases the strength of primary load carrying elements to resist failure under certain specified loading conditions, while designing notional element removal requires a structure to be designed so that it can bridge local failure. The structure is provided with additional strength in areas that are believed to be prone to accidental loads (e.g. exterior columns at risk from vehicular collision) or in key elements that are crucial to the overall structural stability. These members should be able to develop their full resistance against an unanticipated load without failure of either the member itself or its connections. By activating the full resistance available in the key members, this approach maximizes their ability to deal with unforeseen hazards without having to redistribute loads. Due to the unforeseen nature of these events, we cannot accurately predict their magnitude and location. They should be designed for accidental loads, which are specified in several standards as 34 kPa, [17], applied in any direction to the element or attached components (**Figure 4**).

Accidental design situation (ADS) involves exceptional conditions of the structure and its exposure. ADS refer to exceptional conditions applicable to the structure or to its exposure impact or the consequences or localised failure. According to robustness design Codes (BS) situations where damage to small areas of structure or failure of single elements may lead to collapse of major parts of the structure should be avoided. In Designers' guide [34] and in [12] it is specified: each support member should be considered to be nationally removed, one at a time in each storey in turn, and it should be checked that after its removal, the area at risk of collapse of the structure within the storey and the immediately adjacent storeys is limited to: 15% of the of the area of the story, or 70 m² whichever is the less. These limitations are related to the area of risk of collapse in the event of an accident, in plane and section.

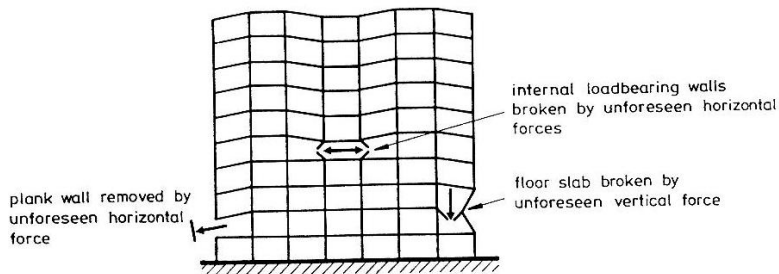


Figure 3. Deferent mechanisms of damages in RC multi-storey building, after [5]

The progressive collapse of RC is initiated when one or more vertical bearing members are removed due to hazards. When any element fails, the remaining elements of the structure seek alternative load path to redistribute the load applied to it. As a result, other elements may fail causing failure mechanism [34], Figure 3.

After the collapse of Roman Point, a fundamental changes in design philosophy of building structures in the UK were made. Recently, in 2004 the Building Regulations for England and Wales were amended again to bring buildings within the scope of the disproportional (progressive) collapse requirements [4]. In the Document there are four building classes: 1, 2A, 2B and 3, depending on type and occupancy Class 1 – includes

houses not exceeding 4 storeys and buildings into which people rarely go. Class 2A (5-storey single occupancy houses, hotels and offices not exceeding 4 storeys, hospitals and industrial building (max 3 storeys), single-storey education buildings, all buildings not exceeding 2 storeys and whose floor area is exceeding 2000 m². Class 2B includes hotels and residential buildings exceeding 4 storeys, but not 15 storeys, hospitals not exceeding 3 storeys, offices 4 – 15 storeys, car parking not exceeding 6 storeys, etc. Class 3 risk assessment includes all the buildings that exceed the limit given in class 2A and 2B.

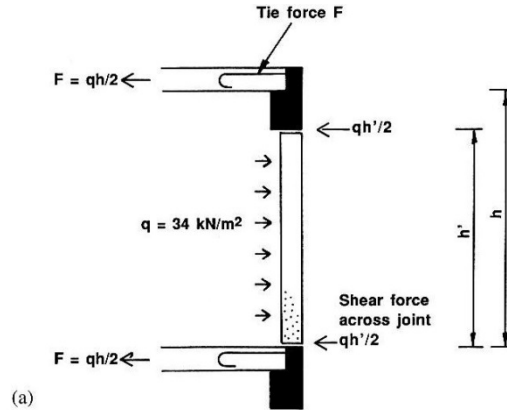


Figure 4. Model for protective member, after [15]

Document NHBC is updated to refer to EN1992-1-1, which contains guidance on the design of ties. Partial factors for accidental situations can be reduced to $\gamma_C = 1.2$ for concrete and 1.0 for steels. At each floor and roof level, an effectively continuous tie should be provided within 1.2 m of the floor edge (**Peripheral ties in floors**) for class 2A and above. The tie should be able to resist a tensile force of:

$$F_{tie,per} = (20 + 4n_0) \leq 60 \text{ kN}, \quad (4)$$

where: n_0 – number of storeys.

Internal ties in floors (for Class 2A buildings and above): In each direction the tie needs to resist a force, which should be taken as:

$$F_{tie,int} = (1/7.5)(g_k + q_k)(\ell_r/5) F_t \geq F_t, \quad (5)$$

where $(g_k + q_k)$ – average permanent and variable floor actions (kN/m²); ℓ_r – greater for distances (in m) between centres of the columns, frames or walls supporting any two adjacent floor spans in the tie under consideration; $F_t = (20 + 4n_0) \leq 60 \text{ kN}$ n_0 – number of storeys.

Horizontal ties to columns and walls (for Class 2A buildings and above): Vertical elements at the edge and corner of the structure should be tied to each floor and roof. In corner columns ties should be provided in two directions. The tie should be able to resist a force of:

$$F_{tie,fac} = F_{tie,col} = \text{Maximum}(\text{Minimum}(2F_t; \ell_s \frac{F_t}{2.5}); 0.03N_{Ed}), \quad (6)$$

where $F_{tie,fac}$ – peripheral tie force (kN/m run of wall); $F_{tie,col}$ – column tie force (kN per column); $F_t = (20 + 4n_0) \leq 60 \text{ kN}$ n_0 – number of storeys; ℓ_s – floor to ceiling height (in meters); N_{Ed} – total design ultimate vertical load in wall or column at the level considered.

Tying of external walls is required only if the peripheral tie is not located within the wall.

Vertical ties to columns and walls (for Class 2A buildings and above):

$$E_d = G_k + A_d + \psi_{1,1}Q_{k,1} + \sum \psi_{2,i}Q_{k,i}, \quad (7)$$

where G_k – characteristic value of a permanent action; A_d – design value a variable action; ψ_1 – frequent value of a variable action; Q_k – characteristics value of a variable action; ψ_2 – quasi-permanent value of a variable action.

In [18] Cl. 9.10.2.5(2) states: “Normally, continuous vertical ties should be provided from lower to the highest level, capable of carrying the load in the accidental design situation, acting on the floor above the column/wall accidentally lost”. Accidental design situation is less than the ultimate limit state combination, and can be determined from table [3] and [4].

In [11] the structure's state was assessed based on the calculated global and inter-storey drifts and the damage coefficient developed. The damage level to the building was established using an integrated approach based on global and inter-storey drifts, so that, depending on the level of displacements for which the drifts are identified, a more reliable answer can be obtained. Applying the damage coefficient, a prompt, reliable and accurate indication can be obtained on the damage level to the entire structure in the capacitive domain, from elastic and non-linear to collapse state. The example of a 10-storey building was used to present the developed procedure which determines adaptability of the system in an incident situation.

The latest nonlinear structural analysis software and the emerging designing methodologies using *Performance-Based Seismic Design* (PBSD), indicate the possibility for reliable and multi-parameter approach in assessing the structure's state. Generally, the level of damage to structures can be assessed based on the following: simplified analysis, analytical procedures, energy criteria, damage index, calculating system performance using *Performance-Based Plastic Design* (PBPD), fragility curves, numerical one step solution, incremental-iterative analysis, *Nonlinear Static Pushover Analysis* (NSPA), *Nonlinear Dynamic Analysis* (NDA), and *Incremental Nonlinear Dynamic Analysis* (INDA) [11].

4. Robustness of precast concrete buildings

Precast structures are more susceptible to the effect of abnormal loads than some traditional forms of construction because of the presence of joints between the structural components. In skeletal structures two greatest problems are: the avoidance of joints in critical locations, and how to design the structure to prevent progressive collapse [14]. In part 7.9 *fib* Model Code 2010 [22] the requirements are shown as follows:

It should be avoided with adequate reliability that accidental and/or exceptional events, or failure of a structural component, cause disproportional damage of a large part of the structure or even total collapse of the whole structure. Hence, the robustness of the system (i.e. adequate safety in relation with accidental and/or exceptional events) shall be analysed. In order to attain adequate robustness of the system one or more of the following strategies may be followed:

- structural measures (the total damage to the system is limited, with sufficient redundancy and possibilities to mobilise the so called alternative load paths);
- limiting the amount of structural damage (by choosing components whose bearing resistance significantly exceeds the expected actions);

- designing the key structural components to withstand every possible load (concept of avoiding actions – prevention, and reactions – protection, and mitigation of consequences);
- non-structural measures (according to the approach directed at avoiding actions);
- reducing the probability that the action occurs or reducing the action intensity (prevention);
- mitigating the consequences of failure.

In the design of conventional structures one might eliminate the use of an explicit robustness requirement and avoid verification of redundancy by taking into the design explicitly as many aspects related to specific accidental actions as possible and applying relevant non-structural measures to deal with them. If robustness requirements are explicitly considered in design, it shall be verified that the structure has sufficient redundancy and possibilities to mobilise alternative load paths. Accordingly, the redundancy of the system shall be verified. For conventional structures, the simplified approach for verification of redundancy is recommended [22]. Often the only rational approach for improving the robustness of an existing system is designing the key structural components so that they can resist any potential load.

For providing robustness by creating an alternative loading path it should be made sure that in the case of an extreme event an alternative loading path can be mobilised around an area of local damage, where the degree of static indeterminacy (number of potential alternative load paths) is important, Cl. 7.9.2 [22]. Two procedures for progressive collapse analysis can be followed to verify the redundancy of a system: a refined procedure and a simplified procedure.

According to the **refined procedure**, the occurrence and the effect of an accidental and/or exceptional action (impact, explosion, etc.) are simulated for all possible action scenarios. The damage of the structural components is calculated and the stability of the remaining structure is assessed. Subsequently, the consequences are estimated in terms of the number of casualties and economic losses. Various measures can be compared on the basis of economic criteria.

According to the **simplified approach**, the situation is considered that any structural component that has significance for the bearing capacity of the structure can lose its bearing function. It is then verified if the remaining part of the structure can withstand the frequently occurring loads with defined target reliability for alternative load paths, for a relatively short period of time, necessary to mobilise the users of the structure and/or to carry out necessary repair or strengthening. For unconventional structures the probability of having some unspecified failure cause is substantial.

This target reliability for alternative load paths depends on the normal safety target for the structure, the period under consideration (hours, days or months) and the probability that the element under consideration is removed (by other causes than already considered in design). For conventional structures it should be possible to include all relevant collapse origins in the design.

Capacity design ensures a hierarchy of resistances of structural components and failure modes that pre-emptive brittle failure modes are prevented and ductile ones promoted. To this end, brittle failure modes are verified for force action effects obtained not by analysis but from equilibrium, assuming that all relevant ductile components develop their force capacities. Ductile structural components are designed then for the plastic deformations induced in them by the design actions.

Capacity design may also be used – in seismic design in particular – to prevent undesirable plastic mechanisms associated with limited global deformation capacity and enforce the intended one(s), involving only ductile structural components.

In a building with a frame structural system a storey-sway mechanism can be prevented and a beam-sway enforced by satisfying all beam column joints conditions:

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb}, \quad (8)$$

where $\sum M_{Rc}$ and $\sum M_{Rb}$ are the sums of the moment resistance of the columns and the beams framing into the joints and $\gamma_{Rd} \geq 1.0$ a factor for overstrengths not explicitly modelled (due to steel strain-hardening) [21 – 22] and [12].

It is important to mention paper [30], which presents the philosophy of designing precast concrete structures under normal and incidental loads. A new approach to structure safety is often called "the philosophy of structural design" [30] because experience has shown that computing limit cases of usability and loadbearing capacity of individual elements and their joints are insufficient to prove a structure's safety. This applies especially in cases of incidental loads such as the impact of strong wind and/or vehicles against the structure, gas explosions, and so on. The remaining structural elements were unable to bridge the damaged area causing a chain collapse (**Figure 5**), which in turn had more severe consequences compared to the direct damage from the gas explosion. Beam actions of walls: Walls may be assumed to be capable of spanning an opening if sufficient tying steel at the top and the bottom of the walls allows them to act as the web of a beam with the slabs above and below acting as flanges.

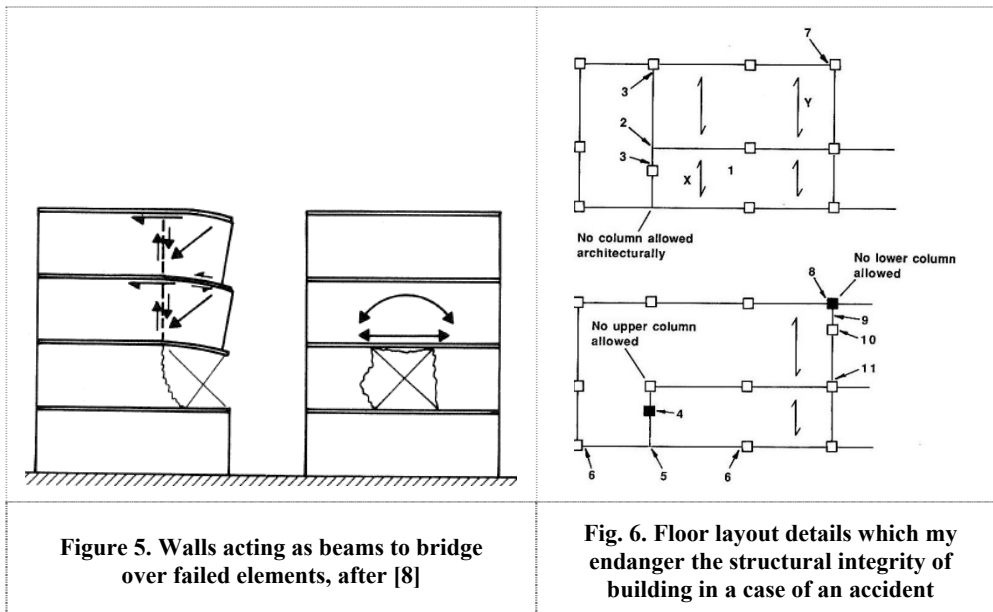


Figure 5. Walls acting as beams to bridge over failed elements, after [8]

Fig. 6. Floor layout details which may endanger the structural integrity of building in a case of an accident

In the precast constructions the layout of the building should be checked to identify any key elements (**Fig. 6**), the failure of which would cause the collapse of more than a limited portion close to the element in question [24]. This key element design must be taken into consideration. Elements other than key elements are provided with vertical ties in accordance with the code provisions. Horizontal ties are divided into floor and beam ties

(Fig. 7). Floor ties provide continuity between floor slabs or between floor slab and beams. Internal and peripheral beam ties provide continuity between the main support beams; thus, gable peripheral beams ties provide continuity between lines of the main support beams.

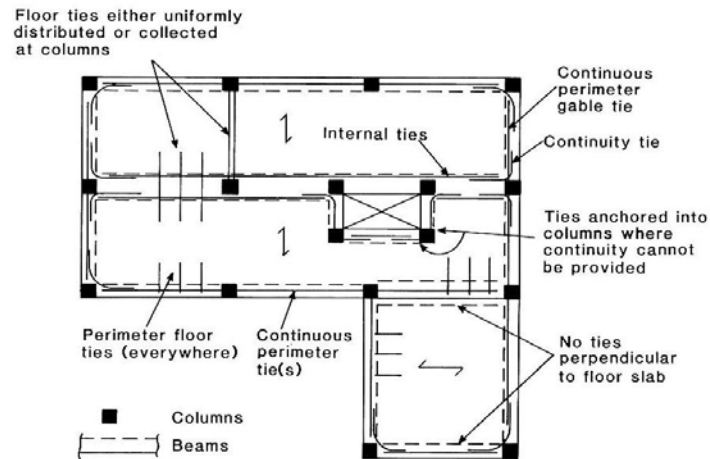


Figure 7. Details and locations of horizontal floor ties in a precast floor, after [14]

Interaction between elements is obtained by the use of horizontal floor and vertical column and wall ties positioned as shown in **Figure 8**. These are as follows: horizontal internal and peripheral ties, which must also be anchored to vertical load bearing elements, vertical ties.

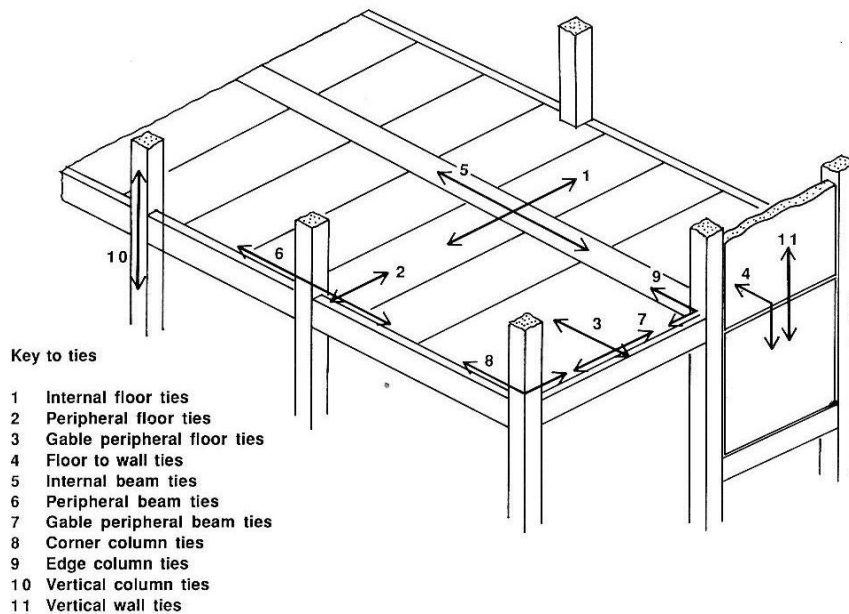


Figure 8. Types of ties in skeletal frames, after [15]

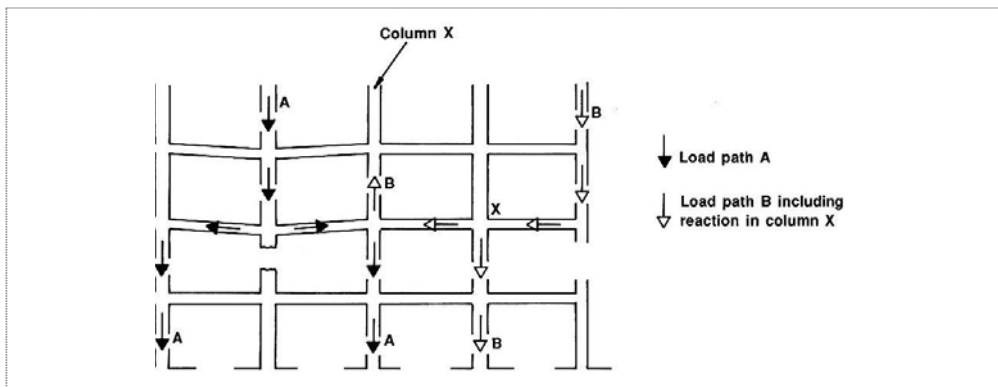


Figure 9a. Alternative load path

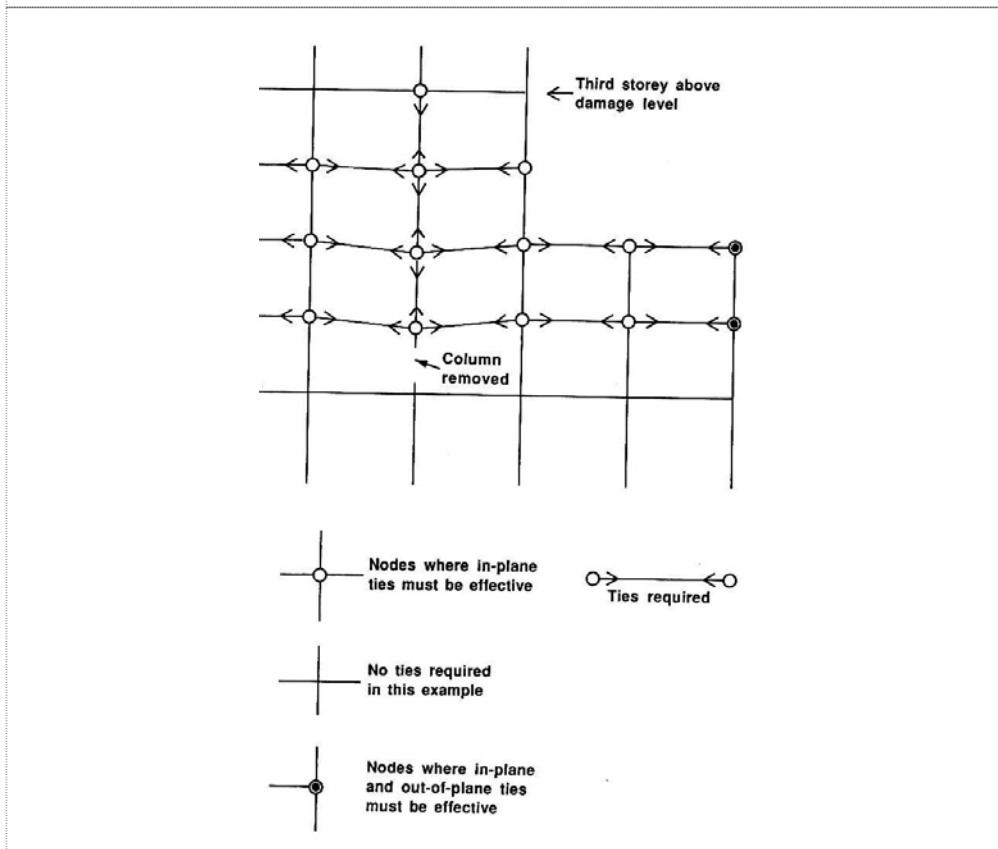


Figure 9b. Use ties for protection

Beside protective members alternative methods are used to design accidental damage shown in [15]: alternative load path (Fig. 9a) and ties (Fig. 9b).

Direct design method, called “alternative path design” specifies walls, beams and columns or part thereof, which are deemed to have failed and the remaining structure is analysed based on these criteria. The components in the remaining structure are called

“bridging elements”. The design [15] should be assured that collapse of a significant part of the structure does not result, as shown in Figure 3.

Redundant structural system: Provide a secondary load path (e.g. an upper level truss or transfer girder system that allows the lower floors of a multi-storey building to hang from the upper floors in emergency) that allows framing to survive removal of key support elements. In the event of the complete loss of a supporting column or beam at a particular floor level, the floor plate is at this level and the next level up must resist total collapse by acting in catenary (= chain link action), as shown in **Figure 10**. If the column is removed, the tie forces over the beams should be mobilised. The column that is directly above the damaged unit carries the beam end reactions of the beams at this level as a tie in vertical suspension. If a beam support is lost, the floor ties act in catenary. If the deflection exceeds, critical value of the tie steel will either fracture or debond in the adjacent spans. Prior to the failure the critical deflection reaches $\sim 0.2 L$ (L -distance between axis of neighbour columns) [14]. The cantilever mechanism is shown in **Figure 11** and discussed in [42] and [45].

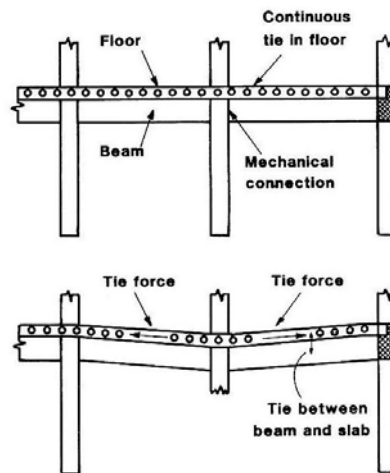


Figure 10. Catenary action between precast elements

Internal ties shall be at each floor and roof level in two directions approximately at right angles. They shall be anchored to the peripheral ties at each end (unless continuing as horizontal ties to columns or walls). The internal ties may, in whole or in part, be spread evenly in the slabs or may be grouped at or in beams, walls or other appropriate positions. Tie bars are either high tensile deformed bar (design strength of 460 MPa) or helical prestressing strand using a design strength about 1580 MPa. The strand lays unstressed, but stretched tightly. Lap length for deformed bar is usually 44 diameters for $f_{cu} = 25$ MPa. A problem occurs at re-entrant corners, shown in **Figure 12a**. The resultant force from the perimeter tie steel is pulling outwards into unconfined space. One cannot rely on the column to restrain this force. Two alternatives are possible as follows: 1) the tie steel continues through or past the side of the column as there was an edge column as shown in **Fig. 12b**. Cast-in (or similar) may be used to anchor the tie steel to the walls; 2) A structural topping containing a steel fabric may be used in this localised area only, but this will probably cause embarrassment to the structural zone in the remainder of the floor. If the floor spans in this part of the building, it can be proportioned so that a shallower precast floor or precast floor or precast beam is used in this area, a topping screed will not increase the overall floor depth. A 50 mm reduction is all that is required [15].

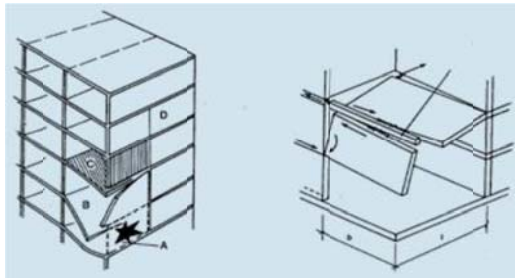


Figure 11. Idealised cantilever action in a wall structure, after SPATeh, after [45]

Where precast floors are designed to act as diaphragms to transfer horizontal forces to bracing elements the following apply: the diaphragm should be adequately reinforced to resist the tension forces in the assumed structural model; where stress concentrations exist in the diaphragm (e.g. around holes, connections to bracing elements), suitable detailing should be provided. Many typical details for continuation are shown in books [14] and [15] and in paper [4]. These details are specifically for robustness using the design forces given in part 3 of this paper. Typical horizontal tie details used for providing opening of adjacent cores at the end supports of hollow core units should be avoided. The recommended maximum length of an open core in hollow core unit is 600 mm. Demountable concrete structures are most often statically determined thus requiring a special treatment for ensuring the stability of such structures [42].

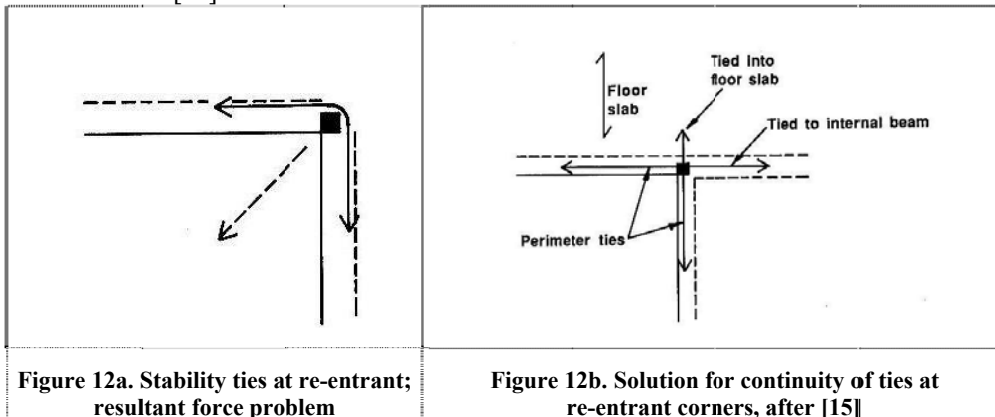


Figure 12a. Stability ties at re-entrant; resultant force problem

Figure 12b. Solution for continuity of ties at re-entrant corners, after [15]

5. Final remarks and conclusions

The load bearing capacity of a building depends not only on the capability of its elements and their joints and connections to receive all influences but also on the way they are spatially connected with tie-beams in the longitudinal, transversal and vertical directions. This is achieved by applying a suitable structural system and appropriate joints. Construction of buildings of different structural systems has different behaviour for accidental actions. Concrete structures of buildings comprise the following types: concrete moment frame, concrete shear wall buildings, dual system - frame with shear walls, and concrete frame with infill masonry shear walls, precast concrete shear wall buildings, precast concrete frame. Buildings are behaving more favourably with stiff floor structures and dual system than with framed structures.

In order to reduce the vulnerability of a structure to disproportionate collapse, one can adopt non-structural protective measures, structural protective measures or a combination of these measures. Progressive collapse as a consequence of local failure has imposed a new philosophy of designing precast concrete building structures [12].

The main objective of the COST Action is to provide the basic framework, methods and strategies necessary to ensure that the level of robustness of structural systems is adequate and sufficient in relation to its function and exposure over its life time and in balance with societal preferences in regard to safety of personnel and safeguarding of environment and economy. It also aims to reduce risks in the built environment in the future.

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