ICOSSAR 2005, G. Augusti, G.I. Schuëller, M. Ciampoli (eds)
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Eurocode 1, Part 1.7, Accidental actions

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Keywords: Accidental action, impact, explosion, risk analysis, robustness

ABSTRACT: Recently the Eurocode on Accidental Actions, officially referred to as EN 1991-1-7, has been completed. The code describes the principles and application rules for the assessment of accidental actions on buildings and bridges. The leading design principle is that local damage is acceptable, provided that it will not endanger the structure and that the overall load-bearing capacity is maintained during an appropriate length of time to allow necessary emergency measures to be taken. As measures to mitigate the risk various strategies are proposed like prevention of actions, evacuation of persons, physical protection of the structure and sufficient structural redundancy and ductility.
The code makes a clear distinction between identified and unidentified accidental actions. For the identified accidental actions (impact, explosions) a structural analysis is proposed, the level of which may depend on the envisaged consequences of failure. It may vary from an analysis on the basis of static equivalent forces to a quantitative risk analysis including nonlinear dynamic structural analysis. Also for unidentified accidental actions the measures depend on the consequence class. In these cases more general measures are proposed to ensure a sufficient robustness of the structure. One may think enhanced redundancy, design of special key elements and three-dimensional tying for additional integrity.
This paper summarises the code and provides some background information as well as design examples.

1 INTRODUCTION

In September 2004, CEN TC250 Subcommittee approved the current draft of Eurocode 1, Part 1.7, Accidental Actions" for a formal voting by the EU-member states. The draft is now in the process of translation into German and French and editorial improvements. This paper summarizes this code and provides some background and examples.

Accidental actions may be defined as actions with low probability, severe consequences of failure and usually of short duration. Typical examples are fire, explosion, earthquake, impact, floods, avalanches, landslides, and so on. Next to these identified accidental actions, structural members may get damaged for a variety of less identifiable reasons like human errors, improper use, exposure to aggressive agencies, failure of equipment, terrorist attacks and so on.

In the Eurocode system, fire and earthquake are dealt with in specific parts. The document EN 1991-1-7 deals primarily with impact and explosion. In addition, the document gives general guidelines on how to deal with identified and unidentified accidental actions in general. The identified actions may be analysed using classical (advanced) structural analysis. For the unidentified actions more general robustness requirements (e.g. prescribed tying forces) have been introduced.

The objective of design in general is to reduce risks at an economical acceptable price. Risk may be expressed in terms of the probability and the consequences of undesired events. Thus, risk reducing measures consist of probability reducing measures (e.g. lightning rods) and consequence reducing measures (e.g. sprinklers, vent openings and so on). No design, however, will be able or can be expected to counteract all actions that could arise due to an extreme cause. The point is that a structure should not be damaged to an extent disproportionate to the original cause. As a result of this principle, local failure may be accepted. For that reason, redundancy and non-linear effects play a much larger role in design for accidental actions than in the case of variable actions.

Design for accidental design situations needs to be primarily included for structures for which a collapse may cause particularly large consequences in terms of injury to humans, damage to the environment or economic losses for the society. A convenient measure to decide what structures are to be designed for accidental situations is to arrange structures or structural components in categories according to the *consequences* of an accident. Eurocode 1991 Part 1.7 arranges structures in the following categories based on consequences of a failure:

class	consequences	Example structures
1	Limited	low rise buildings
2, lower group	Medium	buildings up to 4 stories
3, upper group	Medium	buildings up to 15 stories
4	Large	high rise building, grand stands etc.

Not only the appropriate measures but also the appropriate method of analysis may depend on the safety category, e.g. in the following manner:

-class 1: no specific consideration of accidental actions;

-class 2: a simplified analysis by static equivalent load models for identified accidental loads and/or by applying prescriptive design and detailing rules;

-class 3: extensive study of accident scenarios and using dynamic analyses and non-linear analyses if appropriate.

It is up to the European member states to decide what is considered as an appropriate strategy in the various cases.

2 UNIDENTIFIED ACCIDENTAL LOADS

The design for unidentified accidental load is presented in Annex A of EN1991-1-7. Rules of this type were developed from the UK Codes of Practice and regulatory requirements introduced in the early 70s following the partial collapse of a block of flats at Ronan Point in east London caused by a gas explosion. The rules have changed little over the intervening years. They aim to provide a minimum level of building robustness as a means of safeguarding buildings against a disproportionate extent of collapse following local damage being sustained from an accidental event (Institution of Structural Engineers).

The rules have proved satisfactory over the past 3 decades. Their efficacy was dramatically demonstrated during the IRA bomb attacks that occurred in the City of London in 1992 and 1993. Although the rules were not intended to safeguard buildings against terrorist attack, the damage sustained by those buildings close to the location of the explosions that were designed to meet the regulatory requirement relating to disproportionate collapse was found to be far less compared with other buildings that were subjected to a similar level of abuse.

Annex A, in fact, only specifies operational guidance for consequences class 2. A distinction is made between framed structures and load-bearing wall construction. For framed structures in the lower group of consequences class 2, Annex A recommends horizontal ties in each floor around the perimeter and internally in two right angle directions between the columns (Figure 1). The capacity of the ties is a function of the self-weight, the imposed load and the geometrical properties.

For the upper class 2, in addition one of the following measures should be taken:

- Introduce vertical ties;
- Design key elements for an accidental design action $A_d = 34$ kN/m^2;
- Ensure that upon the notional removal of a supporting column, wall section or beam the damage does not exceed 15% of the floor in each of 2 adjacent storeys.

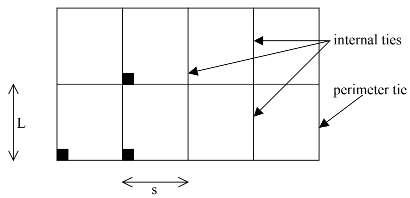

Figure 1: Example of effective horizontal tying of a framed office building.

2.1 *Design example*

As a design example consider a framed structure, 5 storeys with story height h = 3.6 m, consequences class 2, upper group (as originally prepared by the Dutch Studiecel Stufip (2004)). Let the span be L = 7.2 m and the span distance s = 6 m. The characteristic values for the self weight and floor loads are $g_k = q_k = 4$ kN/m^2 respectively and the combination factor $\Psi = 1.0$. In that case, according to Annex A5, the required internal tie force may be calculated from:

$$T_i = 0.8\ \{g_k + \Psi q_k\}\ sL \qquad (1)$$

Leading to:

$T_i = 0.8\ \{4+4\}\ (6 \times 7.2) = 276$ kN

For Steel quality FeB 500 this force corresponds to a steel area $A = 550$ mm^2 or 2 ø 18 mm. The perimeter tie is simply half the value. Note that in continuous beams this amount of reinforcement usually is already present as upper reinforcement anyway. For the vertical tying force we find in a similar way

$T_v = (4 + 4)\ (6 \times 7.2) = 350$ kN/column

This correspondents to $A = 700$ mm^2 or 3 ø 18 mm.

3 IDENTIFIED ACCIDENTAL LOADS

The general principles and combination rules, to be applied in design situations for identified accidental actions, are defined in EN 1990 Basis of Design. Partial load factors to be applied in accidental design situation are defined to be 1.0 for all loads (permanent, variable and accidental).

Combinations for accidental design situations either involve an explicit accidental action A (e.g. fire or impact) or refer to a situation after an accidental event (A = 0). After an accidental event the structure will normally not have the required strength in persistent and transient design situations and will have to be strengthened for a possible continued application. In temporary phases there may be reasons for a relaxation of the requirements e.g. by allowing wind or wave loads for shorter return periods to be applied in the analysis after an accidental event.

Chapter 4 of EC1-Part 1.7 deals with impact from vehicles, ships, trains, fork lift trucks and helicopters. In this paper we will only discuss the impact from vehicles.

3.1 *Impact from vehicles*

The mechanics of a collision may be rather complex. The initial kinetic energy of the colliding object can be transferred into many other forms of kinetic energy and into elastic-plastic deformation or fracture of the structural elements in both the building structure and the colliding object. Small differences in the impact location and impact angle may cause substantial changes in the effects of the impact. This, however, is neglected in Eurocode EN 1991-1-7 and the analysis is confined to the elementary case, where the colliding object hits a structural element under a right angle.

Even then, impact is still an interaction phenomenon between the object and the structure. To find the forces at the interface, one should consider object and structure as one integrated system. Approximations, of course, are possible, for instance by assuming that the structure is rigid and immovable and the colliding object can be modelled as a quasi elastic, single degree of freedom system (see Figure 2). In that case the maximum resulting interaction force equals (CIB, 1993):

$$F = v_r \sqrt{(km)} \qquad (2)$$

where v_r is the object velocity at impact, k the equivalent stiffness of the object and m its mass. In practice the colliding object will not behave elastically. In most cases the colliding object will respond by a mix of elastic deformations, yielding and buckling. The load deformation characteristic may, however, still have the nature of a monotonic increasing function (see Figure 3). As a result one may still use equation (2) to obtain useful approximations. In the background documentation, the model (2) has been successfully compared with experiments by Popp (1961) and Chiapetta and Pang (1981). Also later (unpublished) tests initiated by the British High Way Agency gave results in the same order.

The formula (2) gives the maximum force value on the outer surface of the structure. Inside the structure these forces may give rise to dynamic effects. Amplification factors between 1.0 and 1.4 are considered as a more or less realistic.

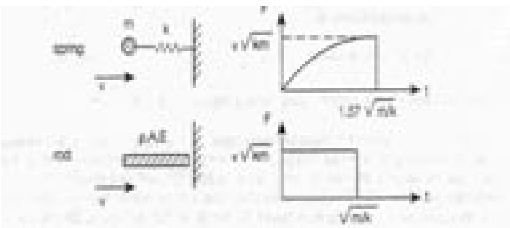

Figure 2 Spring and rod models for the colliding objects

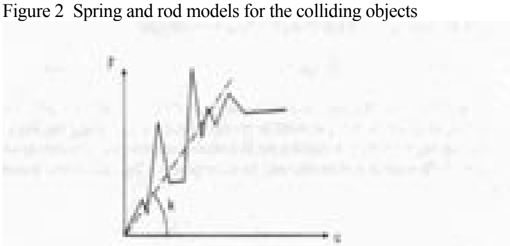

Figure 3: General load displacement diagram of colliding object

The design values in EC1, Part 1.7, Table 4.3, have for political reasons been chosen in accordance with Eurocode 1, Part 3. In the following table some corresponding input values for the parameters m_1, k_o and ν of equation (2) are presented.

Table 1: Calculation of design value of EC1, 1.7.

type of road	mass	velocity	equivalent stiffness	collision force based on (2)
	m [kg]	v [km/h]	k [kN/m]	F [kN]
motorway	20000	40	300	1000
urban area	20000	30	300	500
courtyards - only cars - also trucks	 1500 20000	 5 5	 300 300	 50 150

The input design values for masses and velocity are relatively low values. As a consequence also the F_d value in Eurocode 1, Part 1.7 is low. However, if combined with a conservative linear classic static structural model, the overall design could still very well be over designed.

3.2 *Design example for vehicle impact*

Consider by way of example the reinforced concrete bridge pier of Figure 4. The cross sectional dimensions are b = 0.50 m and h = 1.00 m. The column height h = 5 m and is assumed to be hinged to both the bridge deck and to the foundation structure. The reinforcement ratio is 0.01 for all four groups of bars as indicated in Figure 4, right hand side. Let the steel yield stress be equal to 300 MPa and the concrete strength 50 MPa. The column will be checked for impact by a truck under motorway conditions.

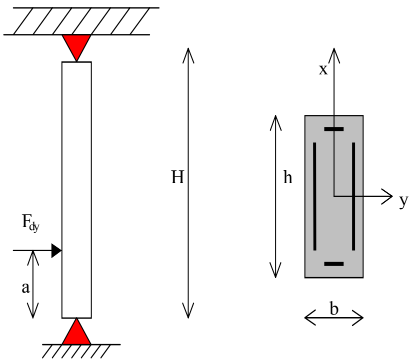

Figure 4 Bridge pier under impact loading

According to the code, the forces F_{dx} and F_{dy} should be taken as 1000 kN and 500 kN respectively and act at a height of a = 1.25 m. The design value of the bending moments and shear forces resulting from the static force in longitudinal direction can be calculated as follows:

$$M_{dx}=\frac{a(H-a)}{H}F_d=\frac{1.25(5.0-1.25)}{5.0}1000=940 \text{ kNm}$$

$$Q_{dx}=\frac{H-a}{H}F_{dx}=\frac{5.00-1.25}{5.00}1000=750 \text{ kN}$$

Similar for the direction perpendicular to the driving direction:

$$M_{dy}=\frac{a(H-a)}{H}F_{dy}=\frac{1.25(5.0-1.25)}{5.0}500=470 \text{ kNm}$$

$$Q_{xy}=\frac{H-a}{H}F_{dy}=\frac{5.00-1.25}{5.00}500=375 \text{ kN}$$

Other loads are not relevant in this case. The self weight of the bridge deck and the traffic loads on the bridge only lead to a normal force in the column. Normally this will increase the load bearing capacity of the column. So we may confine ourselves to the accidental load only. Using a simplified model, the bending moment capacity can conservatively be estimated from:

$$M_{Rdx}=0.8\ \omega h^2\ b f_y =$$
$$= 0.8\ \ 0.01\ \ 1.00^2\ \ 0.50\ \ 300\,000 = 1200 \text{ kNm}$$

$$M_{Rdy}=0.8\ \omega h\ b^2 f_y =$$
$$= 0.8\ 0.01\ 1.00\ \ 0.50^2\ 300\,000 = 600 \text{ kNm}$$

As no partial factor on the resistance need to be used in the case of accidental loading, the bending moment capacities can be considered as sufficient. The shear capacity of the column, based on the concrete tensile part only is approximately equal to (say f_{ctk} = 1200 kN/m2):

$$Q_{Rd} = 0.3\ bh f_{ctk} = 0.3\ 1.00\ 0.50\ 1200 = 360 \text{ kN.}$$

This is almost sufficient for the loading in y-direction, but not for the x-direction. An additional shear force reinforcement is necessary.

3.3 *Explosion loads*

Gas explosions account for by far the majority of accidental explosions in buildings. Gas is widely used and, excluding vehicular impact, the incidents of occurrence of gas explosions in buildings is an order of magnitude higher than other accidental loads causing medium or severe damage (Mainstone (1978), Leyendecker and Ellingwood (1977)).

In this context an explosion is defined as a rapid chemical reaction of dust or gas in air. It results in high temperatures and high overpressure. Explosion pressures propagate as pressure waves. The following are necessary for an explosion to occur (NFPA (1988)):

- fuel, in the proper concentration;
- an oxidant, in sufficient quantity to support the combustion;
- an ignition source strong enough to initiate combustion.

The pressure generated by an internal explosion depends primarily on the type of gas or dust, the concentration of gas or dust in the air and the uniformity of gas or dust air mixture, the size and shape of the enclosure in which the explosion occurs, and the amount of venting of pressure release that may be available. In completely closed rooms with infinitely strong walls, gas explosions may lead to pressures up to 1500 kN/m^2, dust explosions up to 1000 kN/m^2, depending on type of gas or dust. In practice, pressures generated are much lower due to imperfect mixing and the venting which occurs due to failure of doors, windows and other openings.

According to Annex D of EN1991, Part 1.7, elements of a structure should be designed to withstand the effects of an internal natural gas explosion, using a nominal equivalent static pressure given by (Dragosavic (1972, 1973), Leyendecker and Ellingwood (1977)):

$$p_d = 3 + p_v \quad (3)$$

or

$$p_d = 3 + 0.5\, p_v + 0{,}04/(A_v/V)^2 \quad (4)$$

whichever is the greater, where p_v is the uniformly distributed static pressure in kN/m^2 at which venting components will fail, A_v is the area of venting components and V is the volume of room. The explosive pressure acts effectively simultaneously on all of the bounding surfaces of the room. The expressions are valid for rooms up to a volume of 1000 m^3 and venting areas over volume ratios of $0{,}05\ \text{m}^{-1} \leq A_v / V \leq 0{,}15\ \text{m}^{-1}$.

The venting pressure p_v may be determined from tests or may be calculated from uniformly loaded plate bending formulae. Guidance can be obtained in Harris at all (1977).

An important issue is further raised in the note ot clause 5.3.3. It states that the peak pressures in the main text may be considered as having a load duration of 0.2 s. The point is that in reality the peak will generally be larger, but the duration is shorter. So combining the loads from the above equations with a duration of 0.2 s seems to be a reasonable approximation.

The conditions following an explosion, and the reaction of the various elements of the structure to these conditions, are obviously extremely complex. Deciding on a design pressure is only part of the difficulty for a designer. There is then the difficulty of how to assess the probable response of the structure to loading which is short term, dynamic and omnidirectional.

3.4 *Design example for explosion loading*

Consider by way of example a living compartment in a multistory flat building. Let the floor dimensions of the compartment be 8 x14 m and let the height be 3 m. The two small walls (the facades) are made of glass and other light materials and can be considered as venting area. These walls have no load bearing function in the structure. The two long walls are concrete walls; these walls are responsible for carrying down the vertical loads as well as the lateral stability of the structure. This means that the volume V and the area of venting components A_v for this case are given by:

$$A_v = 2 \times 8 \times 3 = 48\ \text{m}^2$$
$$V = 3 \times 8 \times 14 = 336\text{m}^3$$

So the parameter A_v / V can be calculated as:

$$A_v / V = 48 / 336 = 0.144\ \text{m}^{-1}$$

As V is less then 1000 m^3 and A_v / V is well within the limits of 0.05 m^{-1} and 0.15 m^{-1} it is allowed to use the loads given in the code. The collapse pressure of the venting panels p_v is estimated as 3 kN/m^2. Note that these panels normally can resist the design wind load of 1.5 kN/m^2. The equivalent static pressure for the internal natural gas explosion is given by:

$$p_{Ed} = 3 + p_v = 3 + 3 = 6\ \text{kN/m}^2$$

or

$$p_{Ed} = 3 + p_v/2 + 0{,}04/(A_v/V)^2 =$$

$$3+1.5+0.04/0.144^2 = 3+1.5+2.0 = 6.5\ \text{kN/m}^2$$

This means that we have to deal with the latter.

The load arrangement for the explosion pressures is presented in Figure 5. According to Eurocode EN 1990, Basis of Design, these pressures have to be combined with the self weight of the structure and the quasi-permanent values of the variables loads. Let us consider the design consequences for the various structural elements.

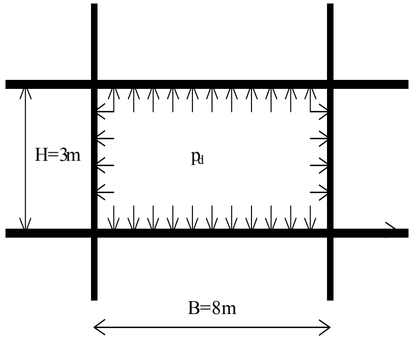

Figure 5: Load arrangement for the explosion load

3.4.1 *Bottom floor*

Let the self weight be 3 kN/m^2 and the characteristic live load 2 kN/m^2. This means that the design load for the explosion is given by:

$$p_{da} = p_{SW} + p_E + \psi_{1LL}\, p_{LL} =$$

$$= 3.00 + 6.50 + 0.5*2.00 = 10.50 \text{ kN/m}^2$$

The design for normal conditions is given by:

$$p_d = \gamma_G\, \xi\, p_{SW} + \gamma_Q\, p_{LL} =$$

$$0.85 * 1.35 * 3.00 + 1.5*2.00 = 6.4 \text{ kN/m}^2$$

We should keep in mind that for accidental actions there is no need to use a partial factor on the resistance side. So for comparison we could increase the design load for normal conditions by a factor of about 1.2. The result could be conceived as the resistance of the structure against accidental loads, if it designed for normal loads only:

$$p_{Rd} = 1.2*6.4 = 7.7 \text{ kN/m}^2$$

So a floor designed for normal conditions only should be about 30 percent too light. It is now time to remember the clause 5.3.3 mentioned earlier. If we take into account the short duration of the load we may increase the load bearing capacity by a factor φ_d given by (see EC1-Part1.7, Background Document (2005)):

$$\varphi_d = 1 + \sqrt{\frac{p_{SW}}{p_{Rd}}}\sqrt{\frac{2u_{max}}{g(\Delta t)^2}} \qquad (5)$$

where $\Delta t = 0.2$ s is the load duration, $g = 10$ m/s^2 is the acceleration of the gravity field and u_{max} is the design value for the midspan deflection at collapse. This value, of course, depends on the ductility properties of the floor slab and in particular of the connections to the rest of the structure. It is beyond the scope of this paper to discuss the details of that assessment, but assume that $u_{max} = 0.20$ m is considered as being a defendable design value. In that case the resistance against explosion loading can be assessed as:

$$p_{REd} = \varphi_d p_{Rd} = [1 + \sqrt{\frac{3}{7.7}}\sqrt{\frac{2*0.20}{10\,(0.2)^2}}]*7.7 = 12.5 \text{kN/m}^2$$

So the bottom floor system is okay in this case.

3.4.2 *Upper floor*

Let us next consider the upper floor. Note by the way that the upper floor for one explosion could be the bottom floor for the next one. The design load for the explosion in that case is given by (upward value positive!):

$$p_{da} = p_{SW} + p_E + \gamma_Q\, \psi\, p_{LL} =$$

$$- 3.00 + 6.50 + 0 = 3.50 \text{ kN/m}^2$$

So the load is only half the load on the bottom floor, but will nevertheless give rise to larger problems. The point is that the explosion load is in the opposite direction of the normal dead and live load. This means that the resistance against the explosion may simply be close to zero. What is needed is top reinforcement in the field and bottom reinforcement above the supports. The required resistance can be found by solving p_{Rd} from:

$$\varphi_d\; p_{Rd} = [1 + \sqrt{\frac{p_{SW}}{p_{Rd}}}\sqrt{\frac{2u_{max}}{g(\Delta t)^2}}]\; p_{Rd} = 3.50$$

Using again $p_{SW} = 3$ kN/m^2, $\Delta t = 0.2$ s, g=10 m/s^2 we arrive at $p_{Rd} = 1.5$ kN/m^2. This would require about 25 percent of the reinforcement for normal conditions on the opposite side.

An important additional point to consider is the reaction force at the support. Note that the floor could be lifted from its supports, especially in the upper two stories of the building where the normal forces in the walls are small. In this respect edge walls are even more vulnerable. The uplifting may change the static system for one thing and lead to different load effects, but it may also lead to freestanding walls. We will come back to that in the next section. If the floor to wall connection can resist the lift force, one should make sure that also the wall itself is designed for it.

3.4.3 *Walls*

Finally we have to consider the walls. Assume the wall to be clamped in on both sides. The bending moment in the wall is then given by:

$$m = 1/16\, p\, H^2 = (1/16)\ 6.5\ 3^2 = 4 \text{ kNm/m}$$

If there is no normal force acting in the wall this would require a central reinforcement of about 0.1 percent. The corresponding bending capacity can be estimated as:

$$m_p = 0.4\, \omega\, d^2 f_y =$$

$$= 0.4\ 0.001\ 0.2^2\ 300.000 = 5 \text{ kNm/m}$$

Normally, of course, normal forces are present. As detailed calculations are out of the scope of this document, the following scheme looks realistic. If the explosion is in a top floor apartment and there is an adequate connection between roof slab and top wall, we will have a tensile force in the wall, requiring some additional reinforcement. In our example the tensile force would be $(p_E - 2\, p_{SW})\, B/2$ = (6.5-2x3) * 4 = 2 kN/m for a centre column and $(p_E - p_{SW})\, B/2$ = (6.5-3) * 4 = 14 kN/m for an edge column. If the explosion is on the one but top story, we usually have no resulting axial force and the above mentioned reinforcement will do. Going further down, there will probably be a resulting axial compression force and the reinforcement could be diminished or even left out completely.

4 CLOSURE

The Eurocode system has been enlarged by an important document on accidental actions. It will be interesting to see how the document will be used in the various member states and how the various national annexes will look like. On almost no other subject the differences between the design rules in the various European member states were so large. This document should for sure be seen as a first step only. Both from a theoretical and a practical point of view improvements need to be made.

REFERENCES

CIB .W81 (1993), "Accidental Actions", Publication 167, CIB, Rotterdam

Chiapella, R.L., Costello, J.F. (1981) "Automobile Impact Forces on Concrete Walls", Transactions of the 6th International Conference on Structural Mechanics in Reactor Technology, Vol. J(b), Paris.

Dragosavic, M. (1972), "Research on Gas Explosions in Buildings in the Netherlands", Notes Reference No. 354/72, Symposium on Buildings and the Hazards of Explosion, Building Research Station, Garston, England.

Dragosavic, M.,(1973) "Structural Measures Against Natural-Gas Explosion in High-Rise Blocks of Flats", Heron, Vol. 19, No. 4 (1973).

Harris, R.J., Marshall, M.R., Moppett, D.J., (1977) "The Response of Glass Windows to Explosion Pressures", British Gas Corporation. I.Chem.E. Symposium Serious No. 49

Institution of Structural Engineers "Safety in tall buildings and other buildings with large occupancy". London, UK.

Leyendecker, E.V. and Ellingwood, B.R., (1977), "Design Methods for Reducing the Risk of Progressive Collapse in Buildings"; US National Bureau of Standards, Washington

Mainstone, R.J., Nicholson, H.G., Alexander, S.J.,(1978), "Structural Damage in Buildings caused by Gaseous Explosions and Other Accidental Loadings, 1971 - 1977", Building Research Establishment, England.

NFPA (1988), "Venting of Deflagrations", National Fire Protection Association

Popp, dr.-ing. Camillo (1961) "Der Querstoss beim Aufprall" (in German)", Forschungshefte aus dem Gebiete des Stahlbaues, Deutschen Stahlbau Verband, Köln am Rhein.

Project Team ENV 1991-2-7, (2005) (under preparation), "Background document to ENV 1991-2-7 Accidental actions due to impact and explosions"

Stufib Studiecel (2004), "Incasseringsvermogen van bouwconstructies" (Robustness of building structures, in Dutch), Delft.

文献紹介

Eurocode 1, Part 1.7, Accidental Actions
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ICOSSAR 2005 in Rome, pp.3311-3317

Eurocode 1, Part 1.7, Accidental Actions が最近最終決定し、現在仏訳よ及び独訳が行われている状態である。この偶発作用に関する設計コードについて、その概要を紹介する。

- 偶発作用は、低い生起確率（質問に対し、構造物の供用期間中に 10-4 以下の生起確率を目安とするという答えだった。）、その影響が大きく、また持続時間が短い作用というのが、一般的な定義である。
- 火災、爆発、地震、衝突、洪水、雪崩、地すべりなどが、含まれる。さらに原因が明確ではない部材の劣化、ヒューマンエラー、不適切な使用、強い作用への曝露、機器の故障、テロ攻撃なども考えられる。
- この中で、Part1.7 は特に衝突と爆発が主な対象である。さらにこのコードでは、衝撃作用を、特定されたもの(identified)と、特定されないもの(unidentified)に分類している。前者に対しては、伝統的な信頼性解析の枠組が、後者に対しては、構造物の強靭性(robustness)などの要求が考慮される。
- 対策としては、リスクの低減（例、避雷針の設置）と被害の低減（例、スプリンクラーの設置）があるが、すべての偶発作用に対処する方法はない。肝要な点は、作用に対して不釣合いに大きな被害を招かないようにするということである。従って構造物の冗長性や非線形性が大きな役割を演じる。
- このコードでは、結果の重大性に基づいて、構造物に範疇を設けている。
  - クラス１　限定的影響　低層建物　偶発作用の影響は特に考慮しない。
  - クラス２　中間的影響　４階までの建物　静的平衡に基づいた検討、慣用的詳細設計の利用など。
  - クラス３　中間的影響　15 回までの建物　偶発作用についても十分に検討する。動的解析、非線形解析の利用も行う。
  - クラス４　重大な影響　超高層建物、競技施設　同上

特定されない(unidentified)偶発作用

この部分は付録 A に記述されており、1970 年代から開発されてきたイギリスのコードを参考にしている。これはロンドン東部の Ronan Point における爆発事件に源を持つコードで、その結果は、1992 年 1993 年の IRA の爆弾事件に対して効果を示した。基本的な考え方は、

構造物の一体性を増すことにより、爆発に対して不釣合いな建物の倒壊を防止するものである。

特定された(identified)偶発作用

特定された偶発作用に乗じる荷重係数は 1.0 である。このとき、永続および変動荷重にかける部分係数も 1.0 である。
Part 1.7 では、自動車、船舶、列車、フォークリフト、ヘリコプターによる衝撃荷重について論じている。

今後は、各国がそれぞれの National Annex で、ここで決められたことをどのように取り扱ってゆくかが興味深い。