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Bases for design of structures — Accidental actions

Bases du calcul des constructions — Actions accidentelles

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Foreword

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The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular, the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

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For an explanation of the voluntary nature of standards, the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the World Trade Organization (WTO) principles in the Technical Barriers to Trade (TBT) see www.iso.org/iso/foreword.html.

This document was prepared by Technical Committee ISO/TC 98, *Bases for design of structures*, Subcommittee SC 3, *Loads, forces and other actions*.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

This document provides requirements and guidelines for the design and assessment of structures in relation to the possible occurrence of accidental actions induced by human activities. Fire and man-made earthquake, however, are not included.

This document is fully aligned with ISO 2394 and gives information for risk informed decision making and semi-probabilistic design and assessment. Like in most modern codes nowadays, attention is given to explicit modelling of hazard scenarios as well as to more implicit safety measurements following from robustness requirements.

This document aims at promoting harmonization of design practice internationally and unification between the respective codes and standards such as for actions and resistance for the respective structural materials.

The principles and appropriate instruments to ensure adequate levels of reliability provide for special classes of structures or projects where the common experience base need to be extended in a rational manner.

The informative annexes included in this document provide support for the interpretation and the use of the principles contained in the normative clauses.

Bases for design of structures — Accidental actions

1 Scope

Accidental actions can be subdivided into accidental actions with a natural cause and accidental actions due to human activities. This document applies to reliability based and risk informed decision making for the design and assessment of structures subject to accidental actions due to human activities. However, fires and human-made earthquakes are not included.

The information presented in this document is intended for buildings and civil engineering works, regardless of the nature of their application and the use or combination of materials. The application of this document can require additional elements or elaboration in special cases.

This document is intended to serve as a basis for those committees that are responsible for the task of preparing International Standards, national standards or codes of practice in accordance with given objectives and context in a particular country. Where relevant, it can also be applied directly to specific cases.

This document describes how the principles of risk and reliability can be utilized to support decisions related to the design and assessment of structures subject to accidental actions and systems involving structures during all the phases of their service life. For the general principles of risk informed design and assessment, it is intended that ISO 2394 be considered.

The application of this document necessitates knowledge beyond that which it contains. It is the responsibility of the user to ensure that this knowledge is available and applied.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the editions cited here apply. For undated references, the latest editions of the referenced documents (including any amendments) apply.

ISO 2394:2015, *General principles on reliability for structures*

ISO 8930, *General principles on reliability for structures — Vocabulary*

3 Terms and definitions

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at http://www.electropedia.org/

For the purposes of this document, the terms and definitions given in ISO 2394 and ISO 8930 and the following apply.

3.1
barriers and shock absorbers
objects or structural devices intended to absorb part of the impact energy in order to protect the structure

3.2
burning velocity
rate of flame propagation relative to the velocity of the unburned dust, gas or vapour that is ahead of it

3.3
deflagration
propagation of a combustion zone at a velocity that is lower than the speed of sound in the unreacted medium

3.4
detonation
propagation of a combustion zone at a velocity that is greater than the speed of sound in the unreacted medium

3.5
dynamic load
time variant load or action that causes significant dynamic effects in the structure or in structural elements

Note 1 to entry: This means that the acceleration is not negligible; as a consequence, equations of motion should be used instead of equations of equilibrium.

Note 2 to entry: In the case of impact, the dynamic load represents a force on an associated contact area at the point of impact.

3.6
equivalent static load
alternative and usually conservative representation of a *dynamic load* (3.6) suitable for a static structural analysis

3.7
explosion
physical and/or chemical process of abrupt release of energy leading to short pressure waves of very high intensity

3.8
flame propagation
speed of a flame front relative to a fixed reference point

3.9
impact
event occurring when one object comes into contact with another one, where the contact force is of short duration

3.10
impacting object
vehicle, ship, etc. colliding with a structure

3.11
key element
structural member upon which the stability of a part of remainder of the structure depends

3.12
local damage
localised failure of a part of a structure that is severely disabled by an accidental event

3.13
unidentified action
accidental action or event that is unknown or unforeseen and cannot be considered by explicit analysis in the design or assessment

3.14
venting panel
non-structural part of the enclosure (wall, floor, ceiling) with limited resistance that is intended to relieve the developing pressure from *deflagration* (3.4) in order to reduce pressure on other parts of the building

4 Symbols and abbreviated terms

4.1 General

The symbols listed in this clause are used generally throughout the document. Symbols which are used only in one section are explained there and not listed here. All the symbols are based on ISO 3898.

4.2 Latin upper case letters

A accidental action, (cross sectional) area

A_d design value of an accidental action

D diameter

E modulus of elasticity, action effect, energy

E_{kin} kinetic energy

E_{def} deformation energy

F action, load in general, collision force

F_R frictional impact force

H height

K_G deflagration index of a gas cloud

K_{St} deflagration index of a dust cloud

L length

P probability

P_f probability of failure

P_{ft} target probability of failure

P_s probability of survival

R resistance

T temperature, period of time

T_e period of time to be considered in a damaged situation

T_{ref} reference period of time

U severity (magnitude) of the source of an action

4.3 Latin lower case letters

a acceleration, geometric parameter

b geometric parameter

c wave propagation speed

f the event of failure, material strength parameter

$f_X(x)$ probability density function of X with dummy variable x

$g(X, t)$ limit state function

h height

h_a height of the application area of a collision force

i impulse per unit of area resulting from explosion

k stiffness

l length

m mass

p momentum (impulse); pressure

p_{stat} static activation pressure that activates a vent opening when the pressure is increased slowly

r distance parameter

r_{F} reduction factor

t time

u displacement;

u_o maximum possible displacement (crumble length of impacting object)

v velocity

4.4 Greek letters

Δ interval

β reliability index

β_{t} target reliability index

ε strain

γ partial factor

γ_{f} partial factors for actions

λ rate of relevant events

μ friction coefficient

ρ mass density

σ stress

4.5 Subscripts

i,j index of basic variable

k characteristic value

d design value

l leading action

max maximum value (often in time)

o initial (reference) value

p plastic

rep representative value

x,y,z coordinate directions

y yield (material)

5 General principles and conceptual approach

5.1 Types of accidental actions

Accidental actions due to human activities shall be considered in the design and assessment of buildings and other civil engineering structures. These actions include but are not limited to:

— Impact from vehicles, trains and tramways, ships, aircrafts, helicopters, forklift trucks, falling materials (rockfall, debris flow, dropped objects from cranes), machine related impacts like toppling cranes, wind turbines, parts detached from a rotary machine, blades detached from turbines, etc.;

— Internal and external explosions due to various sources like gas, dust, TNT, dynamite, etc.;

— Unidentified actions following from:

  — errors in design, errors during construction, service and operation and errors associated with maintenance and repair activities,

  — acts such as sabotage, vandalism, terrorism, etc. and their consequences.

Unidentified actions may be taken into account by specifying the resulting damage to the structure.

Design and assessment decisions related to the occurrence of accidental actions shall be made in accordance with the principles in ISO 2394.

This document shall, for a limited set of relevant actions, provide dedicated information on incident scenarios, load and resistance models, protection systems and calculation procedures.

NOTE Depending on the local circumstances, other actions can also require attention, as for instance avalanches, ice loading, floods resulting from storm surges, heavy rainfall or melting snow, log jams in rivers, sinkholes, etc.

Common impact actions (such as those resulting from stumbling persons, mooring of ships, etc.) should be considered as variable actions and are outside the scope of this document.

The extent and the depth of the design and analysis depend on the possible failure consequences and costs of mitigation.

5.2 Conceptual approach

5.2.1 Target reliability level

The appropriate degree of reliability shall, in accordance with ISO 2394, be selected with due regard to the possible consequences of failure, the associated expense and the level of efforts and procedures required to reduce the risk of failure and damage.

Target reliability levels for existing structures can differ substantially from those for new structures due to economic reasons. Ethical considerations, however, can impose bounds on the outcomes of an economic optimisation.

5.2.2 Strategies

Given the special character of accidental actions, the design approach shall focus on a combination of structural and non-structural measures to either prevent or limit:

— the occurrence of the action;

— the severity of the action;

— the effect of the action in terms of loading on the structure;

— the various direct and indirect consequences.

Direct consequences are damages caused directly by the action; indirect consequences are the result of direct damages, irrespective of the accidental action itself. The ratio between direct and indirect consequences can be seen as a measure of robustness (see ISO 2394).

In many cases, it can be economic, if not unavoidable, to accept some limited degree of direct local damage.

Special devices such as barriers and shock absorbers can be very helpful.

NOTE More information on effects of such devices is presented in Annex C.

5.2.3 Identified and unidentified actions

In the case of identified accidental actions, an assessment on the basis of physical models, reliability considerations and risk analysis shall be performed, depending on the consequence class of the structure.

Since not all possible actions can be foreseen in sufficient detail, the structure shall possess an adequate degree of robustness. In the context of this document, this means that, given the occurrence of local damage or degradation due to an arbitrary accidental action, the probability of a disproportionate collapse should be limited.

5.2.4 Types of analysis

Depending on the function of the structure and the possible consequences in case of failure, the type of analysis and degree of sophistication shall be chosen, both with respect to the physical modelling and to reliability and risk aspects (see 5.3).

The following types of analysis may be used, depending on the applicable risk/reliability aspects (see also ISO 2394):

a) a full risk analysis;

b) a probabilistic analysis based on predefined target reliability levels;

c) semi probabilistic specifications of actions or damage characteristics.

The following types of analysis may be used, depending on the physical modelling (see also 5.4):

— a non-linear dynamic analysis, including load structure interaction;

— a non-linear structural dynamic analysis based on specified external forces or damage characteristics;

— a static structural analysis using quasi static actions or damage characteristics.

Within each of the above analysis categories, further simplifications are possible. The ultimate simplification is to develop a set of prescribed rules. In such a case, the effectiveness of these rules on a global level shall be based on experience (observations), experiments (tests) or advanced analysis procedures. In case of observations and testing, statistical uncertainty as formulated in ISO 2394 shall be accounted for.

Risk and reliability analysis should be based on statistical data as far as possible. Where that is not possible, best estimates based on engineering judgment should be made; these values can also be regarded as nominal values.

5.2.5 Classification of structures based on consequences

The classification system of ISO 2394:2015, Annex F, shall be followed. This system distinguishes 5 classes of consequences, ranging from consequences class CC 1 (predominantly insignificant material damage) to CC 5 (catastrophic losses and large number of exposed persons). The consequence class is in general a useful indicator for both the level of safety measures and the method of analysis to be applied.

5.2.6 Appropriate methods of analyses based on consequences

The extent and the depth of the analysis methods and the appropriate level of mitigation shall be chosen in accordance with the expected consequences.

An appropriate analysis method and level of mitigation shall contain, as a minimum, the following elements depending on the applicable consequence class:

CC 1: No specific consideration of robustness.

CC 2: Simplified analysis based on idealized action and structural performance models and/or prescriptive design/detailing rules.

CC 3: Systematic identification of scenarios leading to structural collapse. Addressing strategies to deal with the identified scenarios. Analyses of structural performance may be based on simplified and idealized models but should be subject to justification. Prescriptive design and detailing rules may be utilized but should specifically address the identified scenarios. Reliability and risk analyses addressing direct and indirect consequences should be used as the basis for simplifications and idealizations.

CC 4: Detailed studies and analyses of scenarios leading to structural collapses utilizing input from experts on all relevant subject matters. Such analyses include detailed assessments using dynamic and non-linear structural analyses and risk analyses rigorously addressing direct and indirect consequences.

CC 5: Same as for CC 4 but with the addition of the involvement of an external expert/review panel for quality control.

From a reliability point of view, simplified models may always be used as long as they are conservative. Whether the degree of conservatism is acceptable or not is an economic issue to be decided by the decision maker.

NOTE The decision maker can be the owner or the competent authority.

5.3 Modelling of accidental actions

5.3.1 Identified actions

The model for extreme hazards such as explosions or collisions resulting in identified accidental actions shall be based on the following:

a) a triggering event at some point in time and place;

b) the amount of energy involved in the event and other relevant parameters;

c) the physical interactions between the event, the environment and the structure, leading to the exceedance of various subsequent limit states in the structure.

All of the above three aspects shall be treated as random quantities and/or random processes as follows:

— The occurrence of the triggering event may often be modelled as events in a Poisson process of intensity λ (t, x) per unit of volume and unit of time, t representing a point in time and x the spatial coordinates (x_1, x_2, x_3).

— The amount of energy may be treated as a random quantity described by a (multidimensional) probability distribution.

— Finally, the physical interactions determining the details of the action and structural response may also be modelled using uncertain variables and properties.

Given these uncertainties, the probability of structural failure (for constant λ and small probabilities) can be expressed as:

$$P_f\left(T_{\text{ref}}\right) \approx \lambda T_{\text{ref}} \int_0^{\infty} P\left(f \mid U=u\right) f_U\left(u\right) \mathrm{d}u \tag{1}$$

where

λ is the number of potential trigger events (e.g. vehicles passing by) per unit of time;

T_{ref} is the reference period under consideration (usually one year or the lifetime of the structure;

f is the failure event to be described by physical models of the structure and the environment;

$f_U(u)$ is the probability density function of the severity (energy) of the hazard, given a trigger event;

U represents the severity (magnitude, amount of available energy) of the hazard;

u is a specific value of U (dummy variable).

The probability of failure can depend on the distance between the structure and the location of the event. In that case, an explicit integration over the area or volume of interest is necessary. If there is more than one hazard, the resulting failure probabilities shall be added, taking into account possible correlations.

Failure in Formula (1) may refer to local or global consequences. The failure probability according to Formula (1) should be less than a specified annual target value, depending on the consequences.

NOTE Target values are usually set between 10^{-6} and 10^{-4} per year (see also ISO 2394).

5.3.2 Unidentified accidental actions

In the case of unidentified accidental actions, the effect of the action shall be modelled as a specific damage (for instance the removal of a specific beam or column). For the remaining part of the structure, for a relatively short period of time T_e (for instance defined as the time to evacuate people out of the building, or the time to repair), the structure shall withstand applicable actions .The corresponding conditional probability of failure shall not exceed a prescribed target reliability, as given by Formula (2):

$$P\left\{R < E \text{ in } T_e \,|\, \text{local damage occurred}\right\} \tag{2}$$

where

R is the resistance of the damaged structure after the occurrence of the unidentified accidental action;

E is the applicable action (effect) after the occurrence of the unidentified accidental action.

The target reliability in this case shall be aligned with the safety target for the building under non-accidental loading, the period T_e under consideration (hours, days or months) and the estimated probability that the local damage under consideration can develop (by causes other than those already considered in design).

NOTE Depending on the circumstances, values between 0,001 and 0,1 can be taken as appropriate.

For unconventional structures (e.g. structures with novel design concepts or using new materials), the probability of having an unspecified cause of failure should be considered as substantial. As a consequence, the target reliability value applicable to Formula (2) sometimes needs to be lowered.

5.3.3 Representative values for accidental actions

Based on the probabilistic approach outlined in 5.3.1 and 5.3.2, appropriate representative values for dynamic or quasi static accidental actions may be derived for use in simplified semi-probabilistic design and analysis.

NOTE Representative values for selected types of accidental actions, based on statistical or other approaches, are presented in Annexes A to C.

5.4 Structural analysis involving accidental actions

The structural analysis involving an accidental action shall, to the extent appropriate for the specific problem, account for:

— severe geometrically nonlinear effects;

— nonlinear material behaviour;

— possible complete rupture of heavily exposed or minor structural elements;

— dynamic effects;

— the interaction between the action and the structure;

— the effects of protecting systems.

Simplified analysis can be appropriate but shall be based on proper justification.

EXAMPLE A quasi static analysis can often replace a full dynamic analysis.

In the case of impact, the most accurate result can be obtained by using an integrated model comprising the impacting body, the structure including the foundation and the protection system if applicable. As a simplification of the analysis, it may be assumed conservatively that the impact energy is fully absorbed either by the structure or by the impacting object.

In the case of an explosion, the action shall be characterized by sudden rises in air pressures and possibly wind effects. The following interaction effects shall be considered:

— the presence of the structure and/or other obstacles that can lead to reflections and turbulence and thus affect the explosion process;

— the collapse of weakened structural elements (accidentally or intended) that can lead to a change of air pressures.

For determining the material properties of the impacting object and of the structure, upper or lower characteristic values should be used, where relevant. Strain rate effects should also be taken into account, where appropriate.

When making simplifications to the analysis, a moderate level of accuracy may be deemed to be sufficient, considering the low probability of the accidental actions.

NOTE For relevant data and approaches of analysis reference is made to Annex C.

6 Impact action

6.1 General

6.1.1 Sources of impact loading

This section applies to the following types of impact actions:

— road vehicles;

— trains and tramways;

— ships;

— aircrafts;

— helicopters;

— forklift trucks;

— falling and sliding materials (e.g. dropped objects, rockfall, debris).

Impact shall be considered in the design and assessment when any of the above (or other relevant) moving objects are in the immediate vicinity of the structure and significant impact forces can occur. The analysis shall be in accordance with the requirements described in 5.2.

Detailed guidance and information are provided in Annex A.

6.1.2 Nature of the impact

During impact, the available kinetic energy of the impacting body shall be considered to be absorbed by deformation of the impacting object itself, deformation of the structure and, if applicable, its protecting systems. Impact interaction actions are usually of high energy and of short duration compared to the longest dynamic natural periods of the structure.

In order to assess the effect of the impact, next to the structural characteristics, the mass, the velocity at the time of impact, the angle of approach and the mechanical properties of the impacting body shall be considered. These parameters usually depend on the characteristics of the environment, such as the type of road or waterway, the distance, local slopes and so on.

Both the likelihood of the initiating event leading to structural damage as well as the statistical characteristics of the colliding object and the structure shall be taken into consideration.

6.1.3 Structural analysis and simplifications

6.1.3.1 In general, collision phenomena involve both the deformation of both the structure and the impacting body. To simplify the analysis, the interaction force may be found by assuming that the structure is fully rigid and the kinetic energy is absorbed and dissipated by the impacting object.

For the accidental type of impact considered in this document, the collision is a process involving (quasi) elastic-plastic deformations. Examples of a rigorous analysis of the collision phenomenon can be found in Annex A. For guidance, the following simplified approaches to estimate the impact forces and durations are recommended:

a) a plastic yield force that is constant with deformation and time;

b) a series of constant forces, each of which having a finite duration;

c) a plastic yield force that varies with deformation and/or time;

d) a quasi-elastic rod or spring model, with calibrated properties.

The first approach is common for simple collisions:

— The primary inelastic response mode and its corresponding plastic, constant resisting force are determined.

— The elastic phase that precedes inelastic phase is disregarded because little energy is absorbed during the elastic phase.

— The time necessary to bring the colliding object to rest is calculated as per Formula (3):

$$\Delta t = m v_r / F_\mathrm{p} \tag{3}$$

where

Δt is the time interval from start of collision to time of zero velocity of colliding object

m is the mass of colliding object

v_r is the velocity of colliding object at the start of the impact

F_p is the plastic resisting force

In order to consider (as in the second approach) a series of constant, but limited in duration, forces, the function needs to be evaluated as per Formula (4):

$$\frac{1}{2}mv_r^2 = \sum F_{pi}u_i \tag{4}$$

where

u_i are the extent of deformations over which the corresponding sequential plastic forces F_{pi} are applicable until the deformation at which all the kinetic energy is dissipated;

F_{pi} are the plastic resistance forces for interval i.

Then the duration of the collision can be estimated approximately by weighting the participation of the various engaged F_{pi}, or by theoretically exact calculations.

The third simplified approach involves postulating a force function that varies with deformation or time. Unfortunately, this approach requires the analyst to develop a forcing function, which can be problematic because the forcing function generally cannot be prescribed in advance. However, some guidance can be developed from the following subclauses and Annex A, where results of some tests and theoretical developments are reported.

The fourth approach may be based on impact by a prismatic and homogeneous elastic rod. Given the primarily plastic nature of the impact process, the elastic model is useful only for the process up to the moment of the maximum indent. The properties of the rod should be considered as quasi-elastic quantities with values calibrated to the real highly non-linear elastic plastic behaviour.

A fully elastic rod develops an internal stress wave that travels from the striking end to the free end, and then back to the striking end, at which point the collision is complete. When using the model to describe the elastic-plastic impact, only the wave travelling from the striking end to the free end is used, neglecting however the reflected wave to simulate the irreversible nature of the deformation. With this model, the maximum resulting dynamic interaction force is given by Formula (5):

$$F = v_r\sqrt{k\,m} \tag{5}$$

where

v_r is the velocity of the rod at impact;

k is the spring stiffness of the rod (i.e., AE/L):

- A is the cross-sectional area of the rod;
- E is the elastic modulus of the rod;
- L is the length of the rod;

m is the mass of the rod.

The travel time of the stress wave from the striking to the free end is L/c_o, where c_o is the rod wave propagation velocity as given in Formula (6):

$$c_o = \sqrt{(E / \rho)} \tag{6}$$

where ρ is mass density.

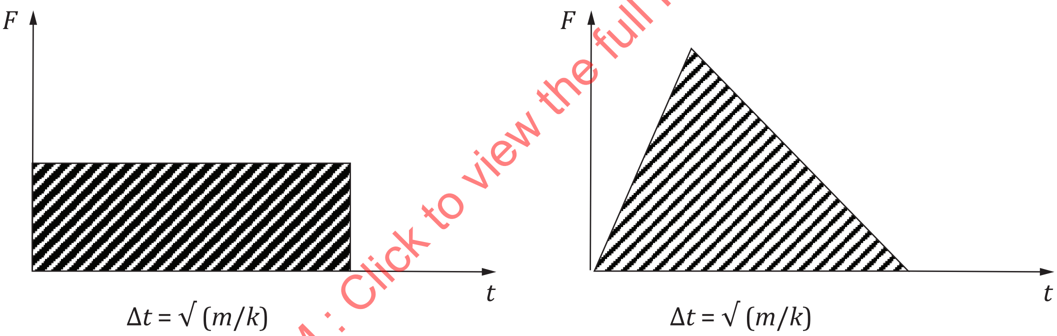
The duration of the collision is given by:

$$\Delta t = L / c_o = \sqrt{(m / k)} \tag{7}$$

NOTE 1 Guidance on appropriate values of k can be found in A.1 to A.7.

NOTE 2 An elastic spring model, with a rigid mass and separate spring with a stiffness constant, k, achieves the same peak force in time as the rod model. However, because the force increases over time, and is not constant as in the rod model, the time to develop the maximum force is: $\Delta t = (\pi/2)\sqrt{(m/k)}$.

NOTE 3 Refined analysis as well as experiments (see Annex A) show that the force time relation can often be represented by a triangle (Figure 1). In that case, the average value of the force follows alternatively from $mv_r/\Delta t$, where estimates for Δt can be found from Formula (7); in absence of other information, the duration of the impact can also be found from $\Delta t = u_o/v$, where u_o is the estimated final indent (often referred to as crumble length). In general, Δt is higher (in the elastic spring model up to a factor $\pi/2$) as the impacting body slows down during the impact process.



Key

F interaction force

t time

Δt time interval

m mass of the colliding object

k stiffness of colliding object

Figure 1 — Schematic pulse shapes in case of a colliding object hitting a rigid, immovable structure

NOTE 4 The models in this clause give dynamic elastic force values on the outer surface of the structure. Within the structure, these forces can give rise to dynamic effects in structural components, depending on the duration of the load. In the absence of a dynamic analysis, the dynamic amplification factor for elastic response can be assumed to be equal to 1,4.

6.1.3.2 An alternative upper bound can be found when the structure dissipates all energy and the colliding object is considered rigid.

If the structure is designed to absorb the impact energy by plastic deformations, provision should be made so that its ductility is sufficient to absorb the total kinetic energy $\frac{1}{2}mv_r^2$ of the colliding object. In the limit case of rigid-plastic response of the structure, the above requirement is satisfied if Formula (8) is verified:

$$(1/2)mv_r^2 \leq F_p\, u_0 \tag{8}$$

where

F_p is the plastic strength of the structure, i.e. the limit value of the static force F;

u_0 is the maximum displacement (deformation capacity) of the point of impact that the structure can undergo, often referred to as crumble length.

NOTE Shear deformation at concentrated forces or near supports is often decisive for the limit on deformation capacity, at least in part because shear failure generally is brittle. When brittle failure modes govern, analysts often increase the assumed impact energy or reduce the crumble length in order to compensate for uncertainties.

In the case of protection by barriers, the formulae should be changed accordingly. See also Annex C.

6.2 Impact from specific causes

6.2.1 Impact from road vehicles

A distinction shall be made between several types of roads, such as highways, local roads, urban roads and parking areas. Each of these road types has a typical distribution for traffic type, traffic intensity, driving velocity, vehicle mass, etc. The traffic type shall also be decisive for the location and size of the impact area on structural elements.

In general, actions in the driving direction (on the traffic way) and the direction perpendicular shall be considered. Road vehicle impact shall be taken into account for road crossing bridges, tunnels and buildings situated close to driveways; in some cases, it is also important to consider vehicles operating inside buildings.

For detailed information on distributions and resulting representative actions, see A.1.

6.2.2 Impact from derailed trains

The effects of derailed trains shall be taken into account for buildings and structures in the vicinity of the railway. Derailing can also affect the superstructure supporting the railroad; effects on the superstructure from derailed rail traffic under or on the approach to a structure need not generally be taken into account as long as no collision with vertical elements is involved.

NOTE Derailment and dislocations of trains on bridge decks are outside the scope of this document.

In general, both loads in the driving direction and in the direction perpendicular to it should be considered.

For detailed information on distributions and resulting representative actions, see A.2.

6.2.3 Impact from ships

Accidental actions due to collisions from ships shall be determined taking account, amongst other factors, the type and dimensions of the waterway, the flood conditions, the type and draught of vessels and their impact behaviour, and the type of the structures and their energy dissipation characteristics.

The effects of hydrodynamic added mass shall be taken into account; in some empirical calculation models, this can already have been done implicitly.

Bow, stern and broad side impact should be considered where relevant. Bow impact should be considered for the main sailing direction with a maximum deviation of 30°.

The action due to impact should be represented by a frontal force, a lateral force with a component acting perpendicularly to the frontal impact force and a friction component parallel to the surface of the structure.

The position and area over which the impact force is applied depend upon the geometry of the structure and the size and geometry (e.g. with or without bulb) of the vessel, the vessel draught and trim, and tidal variations. The vertical range of the point of impact shall account for consistently unfavourable conditions for the vessels sailing in the area.

The forces on a superstructure should be determined by taking account of the height of the structure and the types of ship that can be sailing in the vicinity. In general, the force on the superstructure of the bridge is limited by the yield strength of the ships' superstructure.

Under certain conditions, it can be necessary to assume that the ship is lifted over an abutment or foundation block prior to colliding with columns.

For detailed information on distributions and resulting representative actions, see A.3.

6.2.4 Impact from aircraft

Impact loads from aircraft shall be taken into account for relatively high rise buildings located near airports. Details of the analysis and safety measures to be considered taken should depend on the importance and the strategic value of the building.

For detailed guidance, see A.4.

6.2.5 Impact from helicopters

For buildings with roofs designated as a landing pad for helicopters, an emergency landing (helicopter fall (bad landing) in A.5) force shall be taken into account.

For detailed guidance, see A.5.

6.2.6 Impact from forklift trucks

Impact loads from forklift trucks for buildings in which forklift trucks are operating shall be taken into account.

For detailed guidance, see A.6.

6.2.7 Other types of impact

For many structures, in particular off shore, dropped objects can form a hazard for roofs and pipelines. In addition, blades and loose parts detached from machinery and equipment, snow (avalanches) and falling rocks can cause serious damage. Where relevant, such impact actions shall be considered.

For further infomration, see A.7.

7 Explosion

7.1 General

7.1.1 Explosion types to be considered

Explosions shall be considered in design and/or assessment of buildings and civil engineering works relevant to transport (bridges, tunnels, jetties) where exposure to explosive material and serious consequences can be anticipated.

The origin of an explosion can be of a chemical or physical nature. Chemical explosions can result from:

— natural gas and other explosive gases in buildings (piped gas or in gas in cylinders), tanks or transport vehicles (road tankers, tank wagons and ships);

— dust in buildings;

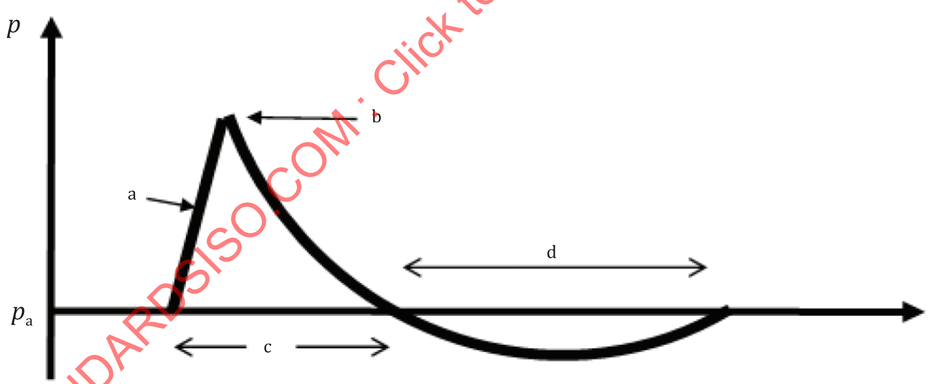
— high energy explosives (e.g. TNT or dynamite in storage or during transport).

An example where an explosion of physical origin can occur is a steam pressure vessel. This type of explosion is considered to be outside the scope of this document.

NOTE A BLEVE (boiled liquid expanded vapour explosion) starts as a physical explosion, but can develop into a chemical explosion.

7.1.2 Nature and schematisation of explosion loading

In an explosion, a very large amount of energy is released in a very short time. The energy can be released via pressure, temperature, radiation and flying debris. Figure 2 gives a characteristic pressure time relation for an explosion.



Key

p pressure

p_a atmospheric pressure

t time

a Rise time.

b Peak pressure.

c Positive phase.

d Negative phase.

Figure 2 — Typical pressure time relation for a vapour explosion

The likelihood of attaining a certain explosion pressure shall be based on:

a) the probability of the presence of explosive material/equipment, its amount and location;

b) the probability of direct or delayed ignition;

d) the characteristics of the structure under consideration.

The following characteristics shall be considered:

— The fuel involved in an explosion can be a combustible gas (or vapour), a mist of combustible liquid, a combustible dust, a high-energy explosive or some combination of these.

— Gaseous fuels have a lower flammability limit (LFL) and an upper flammability limit (UFL). Between these limits for the fuel air ratios, ignition is possible and combustion takes place. The maximum explosion pressure is reached at the so-called stoichiometric mixture.

— The free air pressure following the ignition depends on the amount of available energy and the distance from the location of the explosion. Obstacles in the vicinity of the blast source can have a significant effect on the blast wave in some cases; in particular where the obstacle is situated inside the vapour cloud at the moment of ignition. For explosions inside structures (interior explosions), the venting conditions determine, to a large extent, the final explosion pressures.

— Ignitions of gasses and dust in confined spaces and high-energy explosives normally cause detonations that can be characterized by very rapid pressure increases that decay quickly, and heat-induced expansion of gases that effectively cause “winds”, followed by a lower-intensity negative pressure phase. Ignition of unconfined gas and dust usually results in deflagration, characterized by somewhat lower magnitude positive and negative pressure phases and longer pressure build-up and decay durations. In long tubes like tunnels, conditions can lead to a detonation process where internal ignition is by pressure. This can lead to much higher pressures.

7.1.3 Structural analysis and simplifications

In addition to the general information given in Clause 5, the following analysis features and simplifications shall be applicable to explosions. The positive pressure phase is of interest for most applications and shall be taken into account. The negative pressure phase can be neglected in most applications due to its lower magnitude, and its direction generally acting back toward the explosion point, over areas that have already been exposed to the higher positive pressures. Negative phase loadings can sometimes be important for structures that require elastic response when negative phase coupling with natural modes can be important, and when rebound failures of components are to be avoided.

For many applications, the explosion action can be described with sufficient accuracy by the combination of two parameters: the peak pressure and the duration of the pressure wave or the peak pressure and the impulse.

Explosion pressures may be assumed to be uniform over the length of structural elements. Exceptions occur for detonations near to structural elements, for which there can be significant decreases in pressures at more distant points along a member, and for near-range explosions when the structural element is very near to or within the fire ball of the explosion. In these latter cases, loading histories can be very complex.

Brittle failure modes should be avoided for most explosion resistant designs, since many designs rely on ductility to dissipate energy and compensate for uncertain blast pressures. Non-ductile failure modes, such as buckling and shear at component extremities, should be avoided.

Since the rise time for blast loads is normally much shorter than the longest natural period of structural elements, the load is dynamic or impulsive. Therefore, the use of dynamic analyses to determine structural response should be considered. In some cases, single-degree-of-freedom structural elements subjected to blast loads that are taken to be uniform along their lengths can be assessed following basic theories that predict response as a function of load duration and intensity, natural frequency of the element, and the amount of post-yielding behaviour to be allowed. For other systems, and in particular for multi-degree-of-freedom systems, advanced non-linear computer analyses are usually applied.

7.2 Explosions of various types

7.2.1 Interior explosions

Interior explosions due to dust, gas or vapour shall be considered wherever dust in large quantities can be present and where gases are burned, regulated, transported or stored. The explosion pressure depends primarily on the type of explosive substance, the concentration and uniformity of the dust, gas or vapour air mixture, the ignition source, the presence of obstacles in the enclosure, the size, the shape and the strength of the enclosure and the state of loading in which the explosion occurs, and the amount of venting or pressure release that can be available. The explosive pressure should be assumed to act effectively simultaneously on all of the bounding surfaces of the enclosure in which the explosion occurs.

High energy explosives shall be considered wherever present. When detonated inside structures, they generate very rapid, short-term impulse pressures, followed by build-up of pressurized hot gasses released by the combustion of the explosive material. The geometry of the compartment and the potential for venting through openings and by release of blast panels shall be accounted for in determining the pressure history on any component.

NOTE More information on models for interior explosion loading can be found in Annex B.

7.2.2 Exterior explosion

Exterior explosions can result from transport and storage of explosive materials and shall be considered wherever present in close proximity to a building. It is in particular a common threat to buildings inside or just outside industrial areas that use or manufacture combustible or explosive materials. Exterior explosions also include intentional bomb attacks.

When estimating the pressures, the following shall be taken into account:

— Exterior explosions of gases, vapours and dust generally yield relatively low (but not necessarily negligible) pressures, unless the pre-explosion explosive cloud envelopes a volume that is congested with volumes of components, equipment and mechanical systems. In the latter case, there can be sufficient confinement to yield pressures that tend toward those associated with interior explosions.

— In any case, the magnitudes of the loads depend on the type and concentration of explosive material, the location and configuration of the structure under consideration relative to the explosive cloud or high-energy explosive, and the presence of intervening structure that can impact pressure magnitudes.

NOTE Detailed guidance on models for exterior explosion loading is provided in Annex B.

7.2.3 Explosions in tunnels

Explosions in road tunnels shall be considered when vehicles with dangerous materials like LPG or explosives are to be expected. In tunnels, explosion pressures can become very high and even detonations cannot be completely excluded.

NOTE Detailed guidance on models for explosions in tunnels is provided in B.2.

7.2.4 Dust explosions

Flammable dust explosions shall be taken into account in the design of buildings and other civil engineering works where combustible dust can accumulate on places unlikely to be cleaned in a regular way and ignitions sources can be expected to be present as a consequence of industrial activities.

NOTE Detailed guidance on modelling of dust explosions is provided in B.3.

7.2.5 High energy explosions

High-energy explosions shall be considered in the vicinity of structures where explosives are fabricated, stored, handled and transported.

NOTE Detailed guidance on models of high energy explosions is provided in relevant parts of B.1 and B.4.

8 Unidentified actions

8.1 General

A structure shall be designed against "unidentified" accidental actions, in addition to "identified" accidental actions (see also 5.1). Unidentified actions can include human errors in design, construction and use. Requirements to the response of the structure to unidentified actions should, as far as possible, be similar as to identified accidental actions, that is that damage (or damage increase) is accepted as long it is not disproportionate to the original cause. The process to achieve this is known as design for robustness.

Unidentified actions are intended to be used as a substitute for hazards and situations that cannot be foreseen in detail or at all. For the purpose of this document, unidentified actions shall be considered in one of the two following ways:

— by defining their negative effect on the structure such as a partial or total reduction of the structural capacity of one or more elements as indicated in 8.2;

— by specifying notional actions as indicated in 8.3.

Unidentified actions as provided in this clause may also be used as a simplification of identified actions when a detailed analysis is considered to be unnecessarily laborious.

NOTE 1 The unidentified actions to be taken into account as well as the requirements with respect to the response are to some extent arbitrary and depend on an agreement with the client and regulatory authority. This includes the periods for safe evacuation and rescue operations.

NOTE 2 Detailed guidance is provided in Annex C.

8.2 Notional removal of or damage to elements

As indicated in 5.3 and 8.1, the nature and impact of an unforeseen action cannot be described in detail but, as a meaningful substitute action, a certain degree of reasonable damage may be defined instead.

In particular, the notional removal of a single load bearing member (column, beam or floor) in the structure may be specified. The structure shall then be checked to ensure that upon the notional removal of each element the structure remains stable and that any local damage does not exceed a certain limit.

For buildings that are not frame buildings, the equivalent member to be removed shall be:

— in a reinforced concrete wall, a length not exceeding 2,25 h, where h is the storey height;

— in an external masonry, timber or steel stud wall, the length measured between lateral supports provided by other vertical building components (e.g. columns or transverse partition walls);

— in an internal masonry wall or timber or steel stud wall, a length not exceeding 2,25 h, where h is the storey height.

NOTE These criteria have been taken from Eurocode EN 1991-1-7.

Where the structural form justifies it, more than one member of one type or a combination of types can be removed as notional action. This may be the case if they are very closely spaced such that it is very likely that an unidentified action results in them all suffering damage simultaneously. In the case of a transfer beam, its notional removal requires columns supported by it to be removed simultaneously.

The method of structural analysis used should be commensurate with that used in structural design against identified accidental actions and the method of notional member removal (i.e. whether quasi-static or sudden and causing dynamic effects).

8.3 Notional loads on key elements

As an alternative to the procedure in 8.2, key elements may be designed against the specified notional unidentified actions. The standardized notional action is a uniformly distributed load of 34 kN/m^2.

NOTE 1 This criterion has been taken from Eurocode EN 1991-1-7.

The unidentified action should be applied in the vertical and both horizontal directions (in one direction at a time) to the member and any attached components while giving due regard to the ultimate strength of such components and their connections.

NOTE 2 A key element is a critical member in the absence of which the structure can become damaged to an intolerable extent. In principle, the most obvious critical elements are the columns.

8.4 Risk-based design for unidentified accidental actions

As the unidentified accidental actions are not quantified in intensity and likelihood of occurrence, a risk assessment based on an assumed state of damage should be carried out. This assessment should determine the risks, conditional on a structural element having failed. The tolerable conditional risk shall be greater (e.g. two orders of magnitude) than the value of the unconditional risk chosen for risks corresponding to identified accidental actions.

NOTE 1 A higher tolerable limit can be used for tolerable conditional risk from unidentified actions because the generally low probability of occurrence of the unidentified actions reduces the non-conditional risk. For example, for a probability of occurrence of 1/100 in the period considered (say the life time), the total risk is equal to the unconditional risk divided by 100.

NOTE 2 The tolerable conditional risk level can be determined based on the tolerable unconditional risk level and the potential likelihood of unidentified accidental actions that can be determined based on expert engineering judgment.

For individual projects, communication with local authorities in an early stage of the design is recommended.

Annex A
(informative)

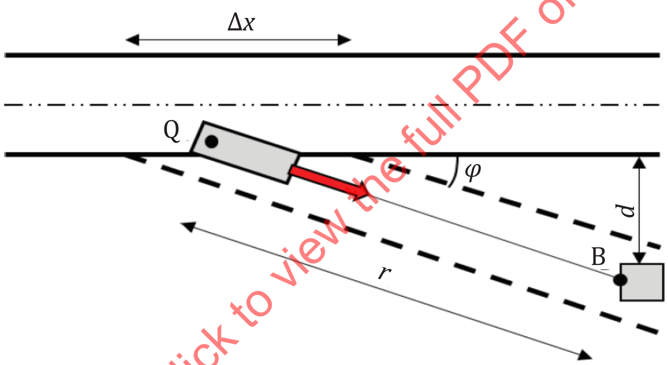
Guidance for detailed impact analysis

A.1 Impact from road vehicles

A.1.1 Impact model

The basic design situation for vehicle impact is given in Figure A.1. The impact force should be determined taking into account the local characteristics of the road, the traffic and the terrain. The object B may be a building, a bridge pier or for instance any protective structure.

A vehicle leaves the intended course at point Q with velocity v_0 and under an angle φ. A structure B or structural member in the vicinity of the roadway at distance r is hit with velocity v_r.



NOTE See Formula (A.1) for designation.

Figure A.1 — Vehicle impact scenario

The probability that there is a collision during a period of time, T, can be approximated by:

$$P_c(T) = n T \lambda \Delta x P\left(v_0^2 > 2ar\right) \quad \text{(A.1)}$$

where

n traffic intensity (depending on local circumstances);

λ vehicle failure intensity (number of road leaving incidents per vehicle km);

T period of time under consideration;

Δx part of the road from where collisions can be expected (see Figure A1);

v_0 velocity of vehicle;

a deceleration of the vehicle after leaving the road;

r $d/\sin\varphi$ = the distance from "leaving point" to "impact point";

d distance from the structural element to the road;

φ angle between vehicle trajectory and road axis.

In case of a lorry impacting a structural member, the velocity v_r at impact may be determined using Formula (A.2):

$$v_r = \sqrt{\left(v_0{}^2 - 2ar\right)} = v_0 \sqrt{\left(1 - d/d_b\right)} \quad (\text{for } d < d_b) \tag{A.2}$$

where

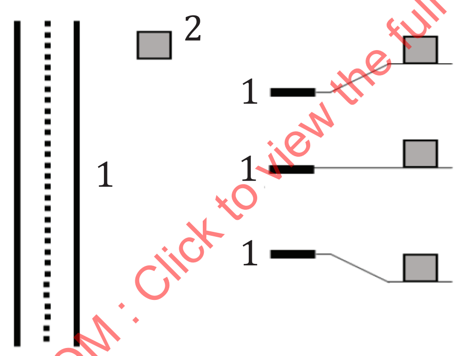
v_0 is the velocity of the lorry leaving the trafficked lane;

a is the average deceleration of the lorry after leaving the trafficked lane;

d_b is the projection of the braking distance r_b : $d_b = r_b \sin\varphi = (v_0/2a) \sin\varphi$.

The value of d_b may be multiplied by 0,6 for uphill slopes and by 1,6 for downhill slopes (see Figure A.2).

The probability of having an impact force F exceeding a value X is the same as in Formula (A.1), but with $P(F(m, k, v_r) > X)$ instead of $P(v^2 > 2ar)$, where for $F(..)$ any suitable impact model may be taken.



Key

1 road

2 structure

Figure A.2 — Uphill and downhill slopes

The traffic intensity and the vehicle failure intensity (number of times a car flies off the road per km of driving) should be estimated on the basis of national or regional statistics, taking care of local circumstances like grades, curvatures, road condition and speed limits. An indicative value for λ on highways is:

$$\lambda = 10^{-7} \text{ per km running of vehicle} \tag{A.3}$$

Further indicative probabilistic information for the basic variables (partly based on statistical data and partly on engineering judgement[1]) is given in Table A.1.

The actual impact on the structure for given values of mass, velocity, etc. follows from advanced or simplified models as described in Clause 6.

Given a target reliability β = 4,0 for a reference period T of 50 years, Table A.2[1] gives recommended design values for some of the input parameters and the resulting impact forces.

Table A.1 — Indicative data for probabilistic collision force calculation

Variable	Designation	Probability distribution	Mean value	Standard deviation
v_o	Vehicle velocity — highway — urban area — courtyard — parking garage	 Lognormal Lognormal Lognormal Lognormal	 80 km/h 40 km/h 15 km/h 5 km/h	 10 km/h 8 km/h 5 km/h 5 km/h
a	Deceleration (on the ground)	Lognormal	4,0 m/s²	1,3 m/s²
m	Vehicle mass — lorry	Normal	20 000 kg	12 000 kg
m	Vehicle mass — car	—	1 500 kg	—
k	Vehicle stiffness	Deterministic	300 kN/m	—
φ	Angle	Raleigh	10° (1:6)	10° (1:6)

Table A.2 — Design values for vehicle mass, velocity and horizontal dynamic impact force F_0

Type of road	Mass m kg	Velocity v_o km/h	Deceleration a m/s²	Impact force[a] F kN	Duration[a] Δt s	Distance[a] d_b [a] m
Motorways	30 000	90	3	2 400	0,3	20
Urban areas[b]	30 000	50	3	1 300	0,3	10
Courtyards — cars only — all vehicles	1 500 30 000	20 15	3 3	120 500	0,1 0,3	2 2
Parking garages — cars only	1 500	10	3	60	0,1	1

[a] Based on Formula (A.2) and Formula (5) with $v_r = v_0$.

[b] Road in areas where the speed limit is 50 km/h.

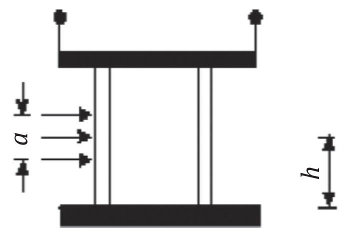
Only almost horizontal terrain is considered.

A.1.2 Practical guidance

A.1.2.1 Impact area

In the absence of further information, the collision force F from lorries may be applied at any height h between 0,5 m to 1,5 m above the level of the carriageway or higher where certain types of protective barriers are provided (see Figure A.3). The recommended application area is a = 0,5 m (height) by 1,50 m (width) or the member width, whichever is the smaller.

For impact from cars, the collision force F may be applied at h = 0,50 m above the level of the carriageway. The recommended application area is a = 0,25 m (height) by 1,50 m (width) or the member width, whichever is the smaller.



Key

a height of the recommended force application area, ranging from 0,25 m (cars) to 0,50 m (lorries)

h location of the resulting collision force *F*, i.e. the height above the level of the carriageway, ranging from 0,50 m (cars) to 1,50 m (lorries)

Figure A.3 — Collision force on supporting substructures near traffic lanes for bridges and supporting structures for buildings

A.1.2.2 Forces on superstructures

Design values for actions due to impact from lorries and/or loads carried by the lorries on members of the superstructure should be defined unless adequate clearances or suitable protection measures to avoid impact are provided. The recommended value for adequate clearance, excluding future re-surfacing of the roadway under the bridge, to avoid impact is in the range of 1 m above the legal limit value.

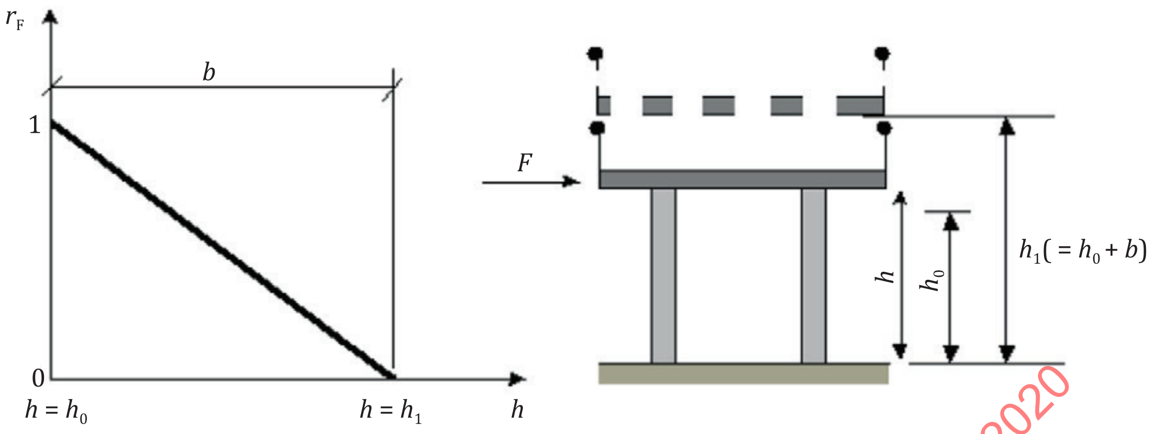
Table A.3 — Indicative equivalent horizontal static design forces due to impact on superstructures

Category of traffic	Equivalent static design force, F_{dx}[a] kN
Motorways and country national and main roads	500
Country roads in rural area	375
Roads in urban area	250
Courtyards and parking garages	75
[a] *x* = direction of normal travel.	

On vertical surfaces, the indicative design impact forces are equal to the equivalent static design forces due to impact given in Table A.3.

For $h_0 \leq h \leq h_1$, these values may be multiplied by a reduction factor r_F. A recommendation is given in Figure A.4.

On the underside surfaces of bridge decks, the same impact forces as above with an upward inclination sometimes have to be taken into account: the conditions of impact may depend on local circumstances. The recommended value of upward inclination is 10^0; see Figure A.5.



Key

r_F multiplication (reduction) factor for the impact force F depending on the value of h

F impact force

h physical clearance between the road surface and the underside of the bridge deck

h_0 minimum height of clearance between the road surface and the underside of the bridge deck below which a full impact on the superstructure needs to be taken into account

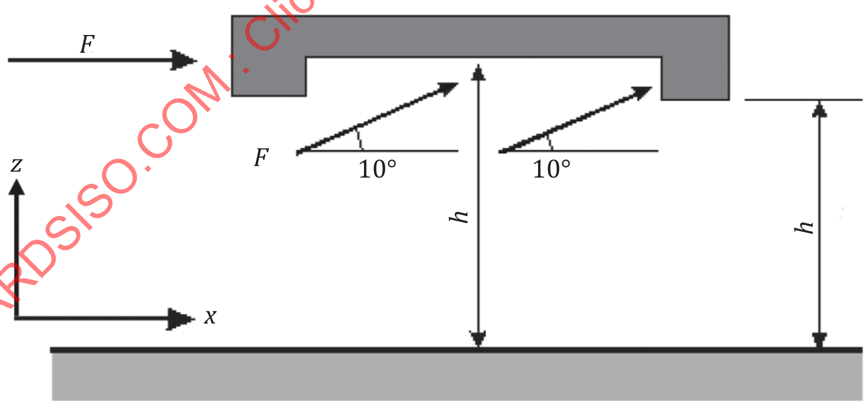
The recommended value of h_0 is the legal truck limit plus 0,1 m.

h_1 maximum value of the clearance between the road surface and the underside of the bridge deck

For values of h_1 and above, the impact force F may be left out of consideration.

b $b = h_1 - h_0$. The recommended value for b is 1,0 m. A reduction factor r_F for F is allowed for values of b between 0 m and 1 m, i.e. between h_0 and h_1

Figure A.4 — Recommended value of the factor r_F for vehicular collision forces on bridge decks



Key

x direction of traffic

z vertical direction

h height of the bridge from the road surface measured to either the soffit or the structural members

F collision forces

Figure A.5 — Impact force on members of the superstructure (y is perpendicular to the paper)

In determining the value of h, allowance should be made for any foreseeable future reduction caused by the resurfacing of the roadway under the bridge.

Where appropriate, horizontal forces perpendicular to the direction of normal travel, F_{dy}, should also be taken into account. It is recommended that these forces do not act simultaneously with the actions resulting from traffic in the driving direction.

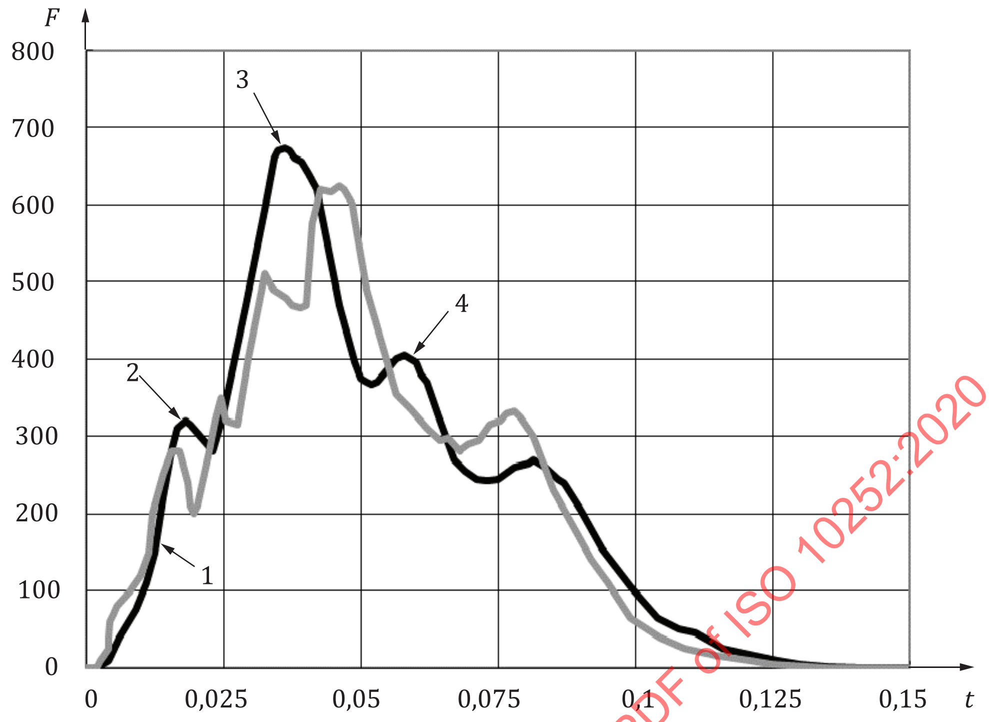
A.1.3 Example of an impact force calculation

A.1.3.1 Impact force versus time based on measured values

For a standard-sized vehicle, the actual impact values measured during the frontal impact tests can be available. The impact force when a standard-sized vehicle of a given weight hits a building with the given impact velocity can be estimated. The load versus time curve in the impact direction obtained from an actual frontal impact test is shown in Figure A.6[2]. The black line in the figure indicates the measured values of the impact force, and the grey line indicates the FEM simulation results of a frontal impact test.

The first rise in the graph is determined by the energy absorbing property of the vehicle. Repetitive interaction among various properties, including deformation, buckling and contact, of the energy absorbers results in deformations of the front part of the vehicle (1 and 2 in Figure A.6). The maximum impact force is observed when there is an impact on the engine (the stiffest structural member) during the event 3. This is the maximum impact force for the building. A deformation around the engine mount follows and the 2nd peak (4) occurs just before the engine comes in contact with the front of the cockpit. The biggest impact shock for the occupant is this 2nd peak.

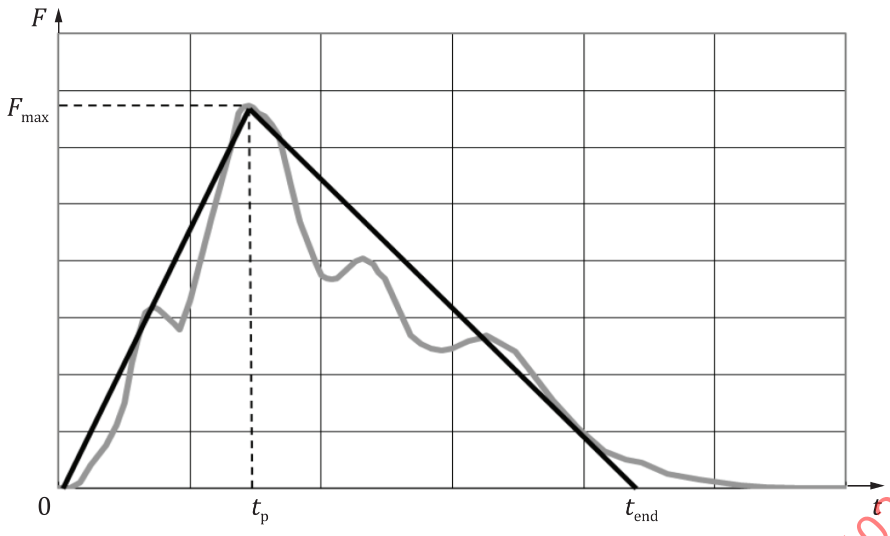
The impact force versus time curve for the frontal impact of a standard-sized vehicle can be approximated to a triangular waveform which peaks during the impact on the engine (Figure A.7). The form of the triangle is determined by the maximum force F_{max}, the peak time t_p, and the acting time t_{end}.



Key

F	force (kN)
t	time (s)
grey line	measured values;
black line	result of FE model simulation
1	The initial stiffness is determined by the stiffness of the energy absorber of the body.
2	The peak due to the behavior (e.g. buckling) of energy absorbing members.
3	The peak due to the impact of the engine.
4	The peak due to the contact between the cockpit and the engine.

Figure A.6 — Example of load versus time curves in the impact direction for the frontal impact test of a standard-sized vehicle[2]



Key

F	force (kN)
t	time (s)
F_{max}	maximum force (kN)
t_p	peak time (s)
t_{end}	acting time (s)

Figure A.7 — Triangular wave approximation of the impact force versus time curve

The estimated force versus time curve for the triangular wave under the given impact conditions is obtained. The results (the maximum force F_{max}, the peak time t_p, the acting time t_{end}) obtained from actual frontal impact tests are shown in Table A.4. The maximum force F_{max} and the peak time t_p can be derived from the measured value in the load versus time curve for the frontal impact test. The acting time t_{end} is determined as an impulse for the load versus time curve (time integral of impact force = acting time t_{end} × F_{max}/2) which corresponds to the momentum p in the impact phenomenon (= vehicle weight m × impact velocity v):

$$t_{end} = 2p/F = 2mv/F \tag{A.4}$$

The relationship between p, F, and t_p is determined by providing p as the indicator of the impact conditions, as shown in Figures A.8 and A.9. The square symbols in the figures indicate the results of the actual impact test and the solid lines indicate the approximated results. The relationship between p and F can be approximated using Formula (A.5). Similarly, the relationship between p and t_p can approximated using Formula (A.6).

$$F = 23,11p + 37,45 \tag{A.5}$$

$$t_p = -3,65 \times 10^{-4}\, p + 3,92 \times 10^{-2} \tag{A.6}$$

where

F is the force given in kN;

p is the momentum in kNs;

t_p is time of the peak in s.

The maximum force is also reasonably well approximated by Formula (3).

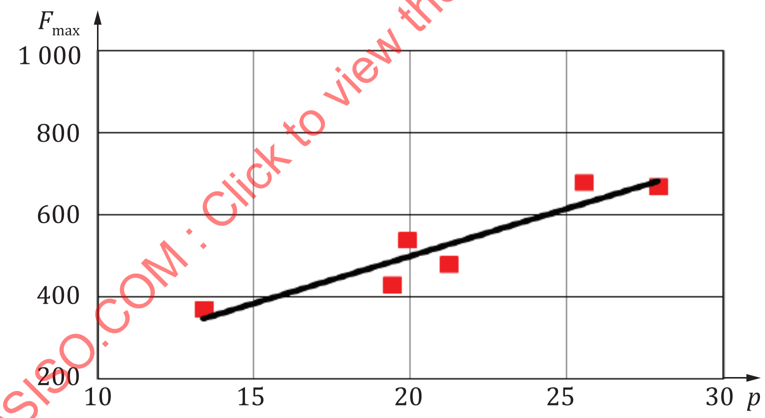
The momentum p is determined, assuming the vehicle mass m and the impact velocity v. The approximation expressions that give the estimated force F, peak time t_p, and acting time t_{end} can be determined using the momentum p as an index for the approximation Formulae (A.4) to (A.6). Finally, these parameters give the approximated impact force versus time curve for a standard-sized vehicle. The acting position of the impact force and the acting area corresponding to the height and width of the vehicle and its relative position to the building can be, consequently, determined.

Table A.4 — Results of frontal impact tests of standard-sized vehicles[2] to [5]

Vehicle	**Vehicle mass** m t	**Impact velocity** v km/h	**Momentum**[a] p kNs	**Maximum force** F_{max} kN	**Peak time** t_p s	**Acting time**[b] t_{end} s
A	1,8	56,2	28	670	0,036	0,083
B	1,2	39,8	13	370	0,038	0,072
	1,3	56,3	20	540	0,031	0,074
	1,3	56,2	19	430	0,031	0,090
C	1,6	56	26	680	0,025	0,075
D	1,4	56,5	21	480	0,028	0,089

[a] Momentum is given by $p=mv$, where m is the mass of the vehicle, v is the impact velocity.

[b] The acting time is given by $t_{end}=2p/F_{max}$.



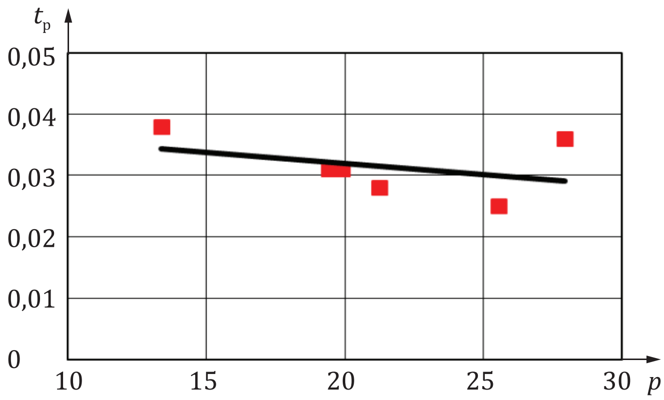
Key

F_{max} maximum force (kN)

p momentum (kN·s)

results of actual impact tests

Figure A.8 — Relationship between the momentum p (kN · s) and the maximum force F_{max} (kN) in a frontal impact test



Key

t_p maximum force (kN)

p momentum (kN·s)

■ results of actual impact tests

Figure A.9 — Relationship between the momentum p (kN · s) and the peak time t_p (s) in a frontal impact test

A.1.3.2 Impact force versus time based on a crash simulation

The following describes the calculation of the impact force from a crash simulation using the finite element method. The comparison between the actual measured values during the frontal impact test of a standard-sized vehicle and its FEM simulation results is shown in Figure A.6. The FEM simulation results are in good agreement with the measured values for the maximum force, peak time and acting time. This proves that the crash simulation with a well-defined finite element model provides an accurate impact force versus time relation for an actual vehicle.

Finite element model of a vehicle was used to obtain an impact force versus time curve. The crash simulation was performed with a finite element model[6][7].

To simulate the behaviour during a hard impact, a frontal crash into a rigid wall was modelled. An accurate finite element model which precisely represents the actual vehicle was used. With the vehicle mass of 1,7 t, three cases with impact velocities of 20 km/h, 40 km/h, and 60 km/h were simulated. The finite element model of a standard-sized vehicle used in the simulation is shown in Figure A.10. A similar example of crash simulations performed using a finite element model of a lorry is shown in References [12] to [13].

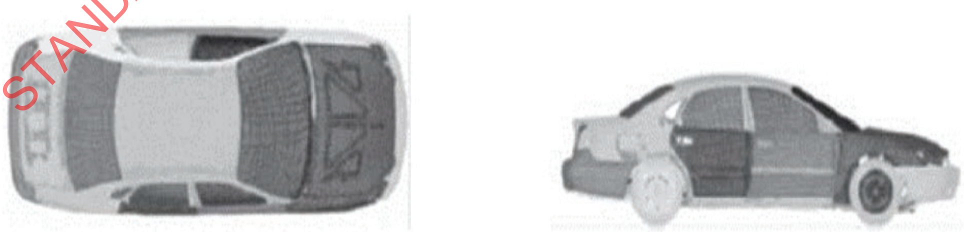


Figure A.10 — Finite element model of a standard-sized vehicle

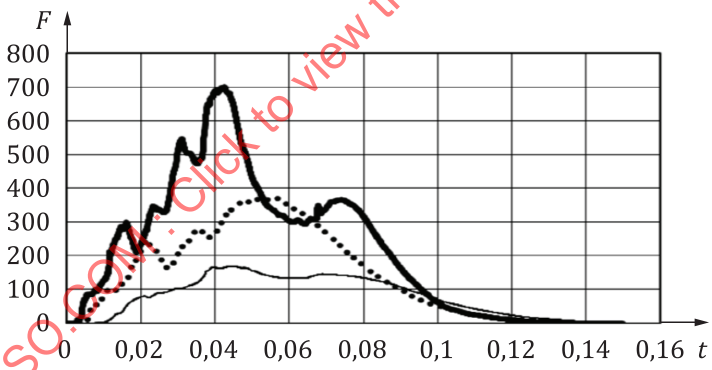
Figure A.11 shows the maximum deformations and Figure A.12 shows the impact velocity versus time curve for each impact velocity. The load values indicate the forces in the impact direction. An accurate finite element model allows the resulted impact force versus time curve to precisely represent the measured values in the experiment, the deformation properties of the structural members and the interactions among structural members (see Figure A.6). The simplified impact force versus time curve from the simulation is shown in Figure A.13 and the corresponding values are shown in Table A.5.

A triangular wave was assumed as the simplified waveform, and the peak time as the time of the maximum force during the simulation. The maximum force and the acting time can be determined to have the impulse of the simplified triangular wave, consistent with the impulse of the simulation result.

The simulated and the measured impact force versus time curves were compared to validate the crash simulation results. The impact force versus time curve based on the measured values is represented using Formulae (A.4) to (A.6) and the estimated results are shown in Table A.6. Figure A.14 shows the comparison between the estimated curve (based on the measured values) and the simplified curve (based on the simulation result). In Figure A.14, the black line indicates the simulation result and the grey line indicates the estimated result based on the measured values. The crash simulation results are more conservative than the estimated results. The acting position of the impact force and acting area are determined corresponding to the height and width of the vehicle and its relative position to the building.

a) Impact velocity 20 km/h **b) Impact velocity 40 km/h** **c) Impact velocity 60 km/h**

Figure A.11 — Maximum deformations for each impact velocity



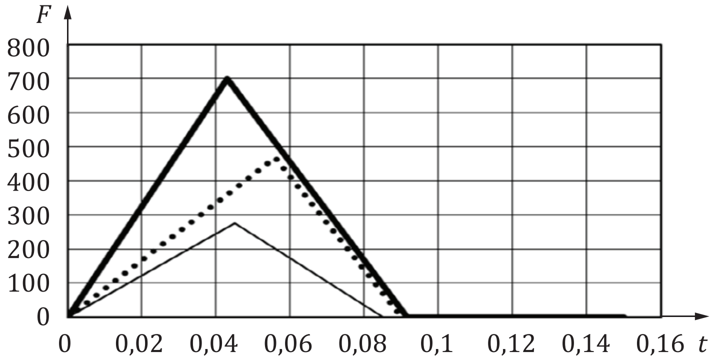
Key

F force (kN)

t time (s)

NOTE From top downward: Impact force versus time curve of the impact velocity 60 km/h, 40 km/h and 20 km/h.

Figure A.12 — Impact force versus time curve for each impact velocity



Key

F force (kN)

t time (s)

NOTE From top downward: Impact force versus time curve of the impact velocity 60 km/h, 40 km/h and 20 km/h.

Figure A.13 — Triangular wave approximation versus time curve for each impact velocity

Table A.5 — Crash simulation results: Maximum force, peak time and acting time of the impact force

Impact velocity km/h	**Impulse**[a] kN·s	**Maximum force**[b] kN	**Peak time**[c] s	**Acting time**[b] s
20	12	275	0,045	0,085
40	21	469	0,056	0,090
60	32	700	0,043	0,092

[a] Time integration value of the force versus time in the impact direction.

[b] The maximum force and the acting time are determined in accordance with the impulse of the simulation result.

[c] The time when the force becomes the maximum in the simulation.

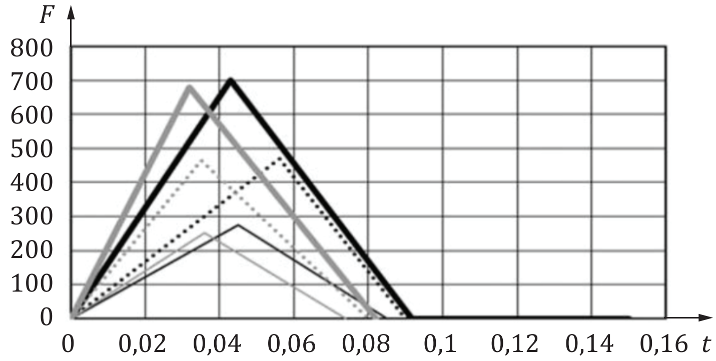
Table A.6 — Estimated results based on measured values

Vehicle mass t	**Impact velocity** km/h	**Momentum** kN·s	**Maximum force**[a] kN	**Peak time**[b] s	**Acting time**[c] s
1,7	20	9	251	0,036	0,074
	40	19	465	0,032	0,080
	60	28	680	0,029	0,082

[a] Calculated by Formula (A.5).

[b] Calculated by Formula (A.6).

[c] Calculated by Formula (A.4).



Key

F force (kN)

t time (s)

NOTE From top downward, 60km/h, 40 km/h and 20 km/h.
Black lines: Values following from FE model simulation.
Grey lines: Estimated values following from measured values

Figure A.14 — Comparisons of triangular wave approximation versus time curve between crash simulation result and estimated result based on measured values

A.2 Impact from derailed rail traffic

A.2.1 Impact model

The collision loads acting on a structure due to derailed trains depend on the mass, collision speed, collision angle, shape, structural strength, stiffness, etc. of the train. The collision loads may be represented using one of the following models:

— as a triangular peak with equal impulse obtained by impact simulation; in some cases, the maximum value of the triangular peak may be used as a static design load; or

— by determining the design loads according to a detailed analytical model.

Impact forces should be determined taking into account the local characteristics of the railroad, the rail traffic types and velocities, traffic intensity and the environmental situation, including protection measures.

The occurrence of a derailment leading to a possible collision with a structural object can be modelled as an (inhomogeneous) Poisson process with failure intensity $\lambda(x)$, in a similar way as for road traffic. This failure intensity, together with other relevant parameters like traffic intensity, should be estimated on the basis of national or regional statistics, taking care of local circumstances like curvatures, switches and grades. If no detailed local information is available, the derailment rate for passenger trains (in the absence of switches) may be taken as[8]:

$\lambda = 0{,}25 \times 10^{-8}$ per km running of train

When switches are present and in particular on emplacements, the value can be higher, up to a factor of 10. For freight trains, derailment rates are a factor 10 higher than for passenger trains.

When calculating the collision load acting on the building, it is necessary to:

a) take into account the different collision parameters, such as the peripheral surrounding situation, for example, the distance between the railway and the building, existence of gravel on the railroad track, protective facilities, etc. of the target buildings and weight and collision speed of the train;

b) assume an undeformable structure such that all kinetic energy at the time of collision is absorbed due to the deformation of the train (see Figure A.17);

c) consider only the lead train car as the collision object. When considering the collision of a train with more than a single car, then the collision of each car should be considered individually or an additional load should be added to the lead car to represent the effect of the remaining cars;

d) consider the train's height, width and position relative to the building when setting the active area and position of the collision load; and assign conservative characteristic values for the physical constants of the structural and non-structural members of the collision body and the building. The strain rate effect chosen should be in the appropriate range.

A.2.2 Practical guidance

This subclause gives practical guidance for design values of the static equivalent forces resulting from the impact of a derailed train with supporting structural members in buildings located adjacent to the tracks or in areas beyond the end of the track[9][10].

By definition according to Eurocode[10], Class A structures either span across or are located near an operational railway. They are either permanently occupied, serve as a temporary gathering place for people, or are more than one story high. For those structures where the maximum speed of rail traffic at the location is less than or equal to 120 km/h, design values for the static equivalent forces due to impact on supporting structural members (e.g. columns, walls) should be specified as shown in Table A.7. The forces F_{dx} and F_{dy}, which are along and perpendicular to the collision direction, respectively, should be applied at a specified height above track level. The design should take $F_{\mathrm{d}x}$ and $F_{\mathrm{d}y}$ into account separately. If the maximum speed of rail traffic at the location is less than or equal to 50 km/h, the values of the forces in Table A.7 may be reduced by 50 %. The duration of the forces is in the order of 0,02 s.

Table A.7 — Indicative horizontal static equivalent design forces due to impact for class A structures over or alongside railways[10]

Distance from structural elements to the centre line of the nearest track d m	Force $F_{\mathrm{d}x}$ [a] kN	Force $F_{\mathrm{d}y}$ [a] kN
Structural elements: $d < 3$ m	— [b]	— [b]
For continuous walls and wall type structures: $3 \text{ m} \le d \le 5$ m	4 000	1 500
$d > 5$ m	0	0

[a] x = track direction; y = perpendicular to track direction.

[b] To be specified for the individual project.

Table A.8 shows the equivalent static force when a derailed train collides with a wall of a structure in an area beyond the end of the track. The loading position is 1,0 m up from the track level.

Table A.8 — Equivalent static force acting on the structure in areas beyond the end of the track

Types	Force $F_{\mathrm{d}x}$ kN
Passenger train	5 000
Multicar train after shifting to a branch line	10 000

A.2.3 Example calculation of the collision load

An example calculation of the collision loads from rail traffic is described here according to the AIJ design guideline[12][13].

A finite element model of a train developed by Hisamori and Tachibana[11] was used to calculate the collision load. The train was made of stainless steel. Its total length and weight were 20 m and 26,3 t, respectively. The finite element train model is shown in Figure A.15. The analysis was performed by assuming a frontal collision with a rigid wall. The collision was simulated as a hard impact[7]. Four different collision speeds were considered in the analysis: 20 km/h, 40 km/h, 60 km/h and 80 km/h.

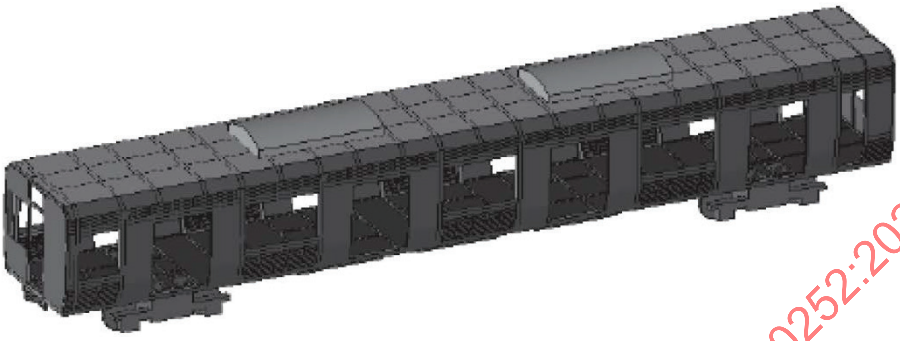


Figure A.15 — Finite element model of the train[11]

Figure A.16 a) shows the analytical results for the time history of the collision load at each collision speed. Figure A.17 shows an example of the maximum deformation of the train obtained by the analytical results for the collision speed of 60 km/h. Figure A.16 b) shows simple triangular peaks chosen to approximate the time history of the collision load. The principle deformation of the train is a buckling of its front end. When the collision speed is increased, the deformation progresses to the rigid foremost wheel part, and the maximum load value is obtained when the collision with the wheel occurs. The parameters of the simple triangular peaks are shown in Table A.9. As the analysis results for collision speeds of 60 km/h and 80 km/h show, a single triangular peak is not always a good approximation of the actual time history of the collision load. For these cases, the parameters of the triangular peak approximation were determined using the maximum load value from the analytical results and the load gradient near the maximum load in accordance with the following:

a) For collision speeds of 60 km/h and 80 km/h, the second peak in the analytical result for the time history of the collision load occurs when the deformation reaches the wheel. The maximum value for the second peak is taken to be the maximum load in these cases;

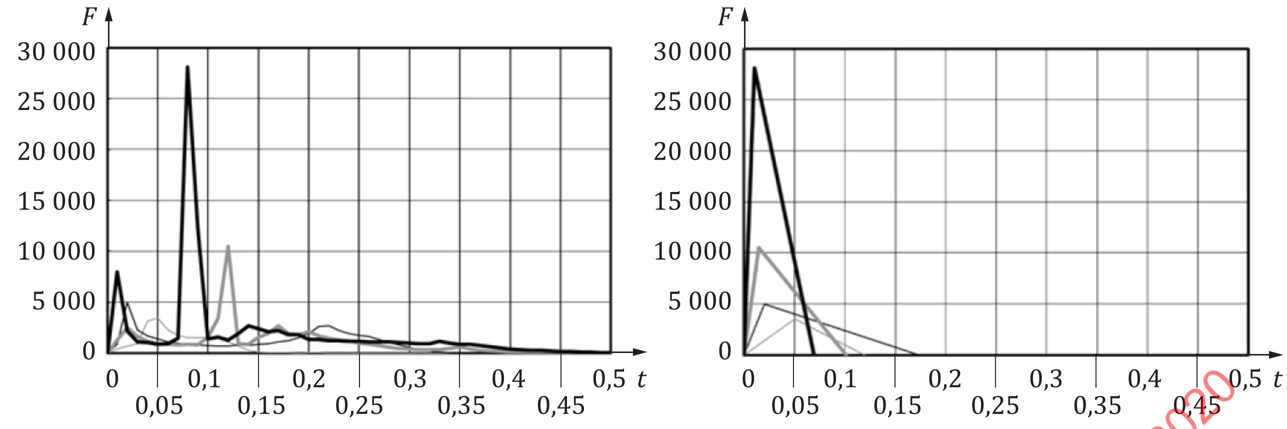
b) It is assumed that the maximum response of the building is due to the maximum load. Therefore, the maximum value of the triangular peak is set to the maximum load in the analytical result;

c) The peak time for the triangular peak is chosen to reproduce the load gradient near the maximum load. The peak time is obtained from the calculation result by dividing the maximum load value by the load gradient near the time of the maximum load;

d) It can be assumed that the maximum response and dynamic effect of the building can be reproduced by introducing the maximum load and the load gradient near the maximum load.

The parameters of the triangular peak are chosen so that its impulse is equal to the impulse of the analytical results. The acting position and area of the collision load are set up in consideration of the train height, the train width and the relative position between the building and the train.

The collision load evaluation using the detailed three-dimensional train model has been performed in recent years with the improvement of computer performances[14][15].



a) **Time history of the collision force based on simulation**

b) **Triangular peaks used to approximate the simulation results**

Key

F force (kN)

t time (s)

NOTE From top downward, 80 km/h, 60 km/h, 40 km/h and 20 km/h.

Figure A.16 — Time history of the collision force at each collision speed

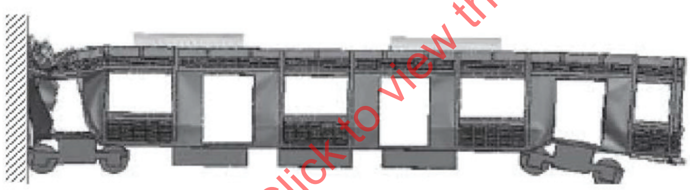


Figure A.17 — Impact simulation (the maximum deformation)[11]

Table A.9 — Parameters for simple triangular peak used to approximate the time history of collision force at each collision speed

Collision speed v km/h	Maximum force F_{max} kN	Peak time[b] t_p s	Duration time[a] t_{end} S
20	3 500	0,05	0,12
40	5 000	0,02	0,17
60	10 500	0,015	0,10
80	28 100	0,011	0,07

[a] Duration time was chosen so that the impulse of the triangular peak approximation is equal to the impulse of the analytical results.

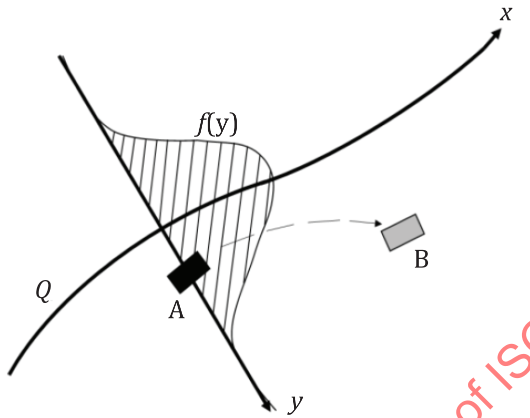
[b] The peak time is the same as the time of the maximum load in the analysis results. For the cases when the collision velocity was 60 km/h and 80 km/h, the peak time was chosen to reproduce the load gradient near the maximum load of the analysis results (peak time = maximum force / force gradient at a time near the maximum force).

The maximum forces in Table A.9, calculated as a frontal collision by assuming an undeformable structure, are conservative evaluation values and are larger than the values in Table A.8[Eurocode]. When using them as an equivalent static force, it is necessary to judge the evaluation method according to the collision situation.

A.3 Impact from ships

A.3.1 Impact model

Impact forces for the situation shown in Figure A.18 should be determined taking into account the local characteristics of the waterway, the ship types and velocities, traffic intensity and the environmental situation (open water, rivers, canals, curvatures, harbour, sluices).



Key

1 ship
2 structural object
x centre line coordinate of water lane
y coordinate perpendicular to centre line
A ship position at the time of fatal error
B structure
$f(y)$ distribution of ship position in sideway direction at the time of the fatal error

NOTE The scenario is as follows: a ship follows with some random sideway deviation the basic course line when a fatal error occurs to the ship A at position (x, y); as a result, there is a collision with structure B.

Figure A.18 — Ship collision scenario

The occurrence of a mechanical or navigation error, leading to a possible collision with a structural object, can be modelled as an (inhomogeneous) Poisson process. Given this Poisson failure process with intensity $\lambda(x)$, the probability that the structure is hit at least once in a period t can be expressed by Formula (A.7).

$$P_{\text{col}}(T_{\text{ref}}) = n\,\lambda\,T_{\text{ref}}\,(1-P_{\text{a}})\iint P(\text{collision}\,|\,v,x,y)\,f(y)\,\text{d}y\,\text{d}x \tag{A.7}$$

where

n is the number of ships per time unit (traffic intensity);

λ is the probability of a mechanical failure or human error per unit of travelling distance;

T_{ref} is the reference period (e.g. 1 year);

P_{a} is the probability that a collision is avoided by positive human intervention;

x, y are the coordinates of the point of the fatal error or mechanical failure;

v is the velocity at point (x, y);

$f(y)$ is the density function of the position distribution in y-direction at point of critical failure.

The probability of failure per unit of distance can depend on technical failures (engine, rudder and communication), human errors and weather condition (wind, rain, visibility). A standard value for hitting a side object can be (informal estimation based on Dutch observations):

$$\lambda = 10^{-6} \text{ per km sailing of ship} \tag{A.8}$$

A.3.2 Elaboration for inland waterways

A.3.2.1 Ship mass

Relevant ship masses depend to a great extent on the type of waterway. Classification systems can be helpful in determining the ship masses that can be expected near the structure to be assessed. An example according to the Conférence Européenne des Ministres de Transport (CEMT) is presented in Table A.10.

Table A.10 — CEMT classification of inland ships

Ship class	Length m	Width m	Depth m	Mass t
0	<30	—	—	<250
I	30 to 50	5,05	1,80 to 2,20	250 to 400
II	50 to 60	6,60	2,50	400 to 650
III	60 to 80	8,20	2,50	650 to 1 000
IV	80 to 90	9,50	2,50	1 000 to 1 500
Va	90 to 110	11,40	2,50 to 2,80	1 500 to 3 000
Vb	110 to180	11,40	2,50 to 4,50	3 000 to 6 000
Via	110 to 180	22,80	2,50 to 4,50	3 000 to 6 000
VIb	110 to 190	22,80	2,50 to 4,50	6 000 to 12 000
Vic	190 to 280	22,80	2,50 to 4,50	10 000 to 18 000
VII	300	34,20	2,50 to 4,50	14 000 to 27 000

A.3.2.2 Impact velocity

If no specific local data is available, mean values of 1,5 m/s for loaded ships and 2,0 m/s for unloaded ships can be considered. In harbour areas and in the vicinity of sluices, the velocities may be reduced to 1,0 m/s and 1,5 m/s. A lognormal distribution with v = 0,5 is recommended. Corrections can be needed for currents and in case of drifting of unpowered ships.

A.3.2.3 Impact forces

Given the impact energy in MNm, the following value for the elastic impact forces (in MN) is recommended[16]:

$$F_{\text{dyn,el}} = 10{,}95\sqrt{E_{\text{imp}}} \qquad [\text{MN}] \tag{A.9}$$

For the plastic stage (E_{imp} > 0,21 MNm):

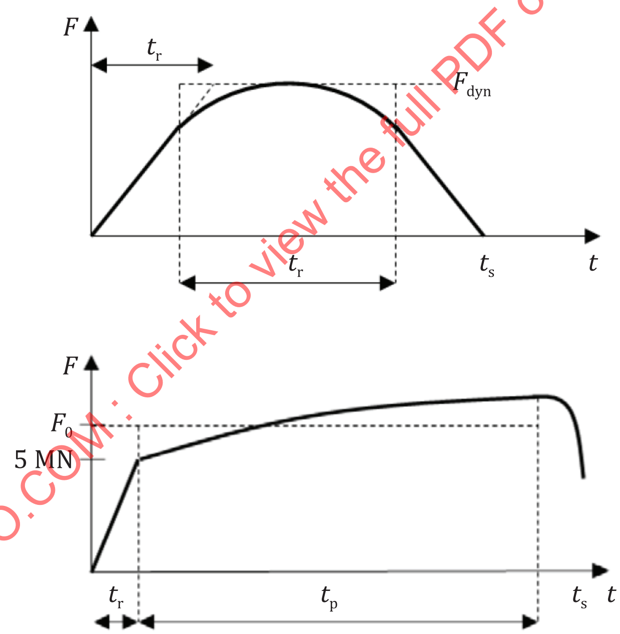
$$F_{\text{dyn,pl}} = 5{,}0\sqrt{\left(1 + 0{,}128 E_{\text{imp}}\right)} \qquad [\text{MN}] \tag{A.10}$$

The impact energy E_{imp} is equal to the available kinetic energy $E_a = 0{,}5\, mv^2$ in case of a frontal collision.

In case of a lateral impact with an angle between ship direction and object surface α < 45°:

$$E_{\text{imp}} = E_a\,(1 - \cos\alpha) \tag{A.11}$$

The load duration and other details may be derived from Figure A.19, using the conservation of momentum.



Key

t_r elastic period, in s;

t_p plastic period in s;

t_e elastic response time, in s;

t_a equivalent impact duration, in s;

t_s total impact duration, in s; $t_s = t_r + t_p + t_e$;

c elastic ship stiffness (= 60 MN/m);

F_0 elastic-plastic limit (= 5 MN);

x_e maximum elastic deformation (≈ 0,1 m);

v_n sailing velocity v_r, for frontal impact, $v_r \sin\alpha$ for lateral impact.

Figure A.19 — Load-time function for ship collision for elastic (upper figure) and plastic impact (lower figure)

The mass m to be taken into account is the total mass of the colliding ship/barge for frontal impact and $(m_1 + m_{hydr})/3$ for lateral impact with m_1 the mass of the colliding ship or barge and m_{hydr} the hydraulic added mass. For the hydrodynamic mass, values of 10 % of the mass of displaced water for bow and 40 % for lateral impact are recommended.

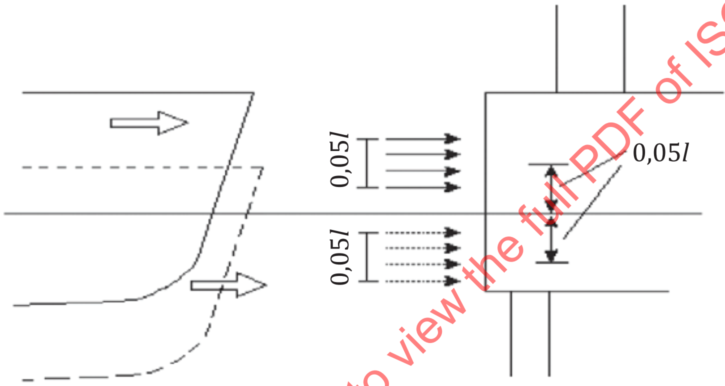
In the absence of detailed analysis, example values of the forces due to ship impact on inland waterways given in A.3.4 may be used.

A.3.2.4 Impact area

The position and the area of impact depend on the geometry of the structure and the ship. In particular, where relevant, the presence of a bulb front should be taken into account (see Figure A20).

A.3.2.5 Loads on the super structure

A range of 5 % to 10 % of the bow impact force may be considered as a guideline. In cases where only the mast is likely to impact on the superstructure, the indicative design force is 1 MN.



NOTE The height of the area is 0,05 *l* but it may be located fully below the low or above the nigh water level. See EN 1991-1-7.

Figure A.20 — Indicative impact areas for ship impact

A.3.3 Elaboration for seagoing vessels

A.3.3.1 Ship mass

The distribution of ship masses should be derived on the basis of the local circumstances. The values presented in Table A.12 can be helpful.

A.3.3.2 Impact velocity

When not specified by the project, a design velocity v_{rd} equal to 5 m/s increased by the water velocity is recommended; in harbours, the velocity may be assumed as 2,5 m/s.

A.3.3.3 Impact force

Given the impact energy, the force may be calculated from Formulae (A.12) and (A.13):

$$F_{bow} = F_0 \underline{L} \sqrt{\underline{E}_{imp} + (5 - \underline{L}) \underline{L}^{1,6}} \quad \text{for } \underline{E}_{imp} > \underline{L}^{2,6} \tag{A.12}$$

$$F_{\text{bow}} = 2{,}24 F_0 \sqrt{\left(\underline{E}_{\text{imp}} \underline{L}\right)} \quad \text{for } \underline{E}_{\text{imp}} < \underline{L}^{2{,}6} \tag{A.13}$$

where

$\underline{L}$ $= L_{pp}/275$ [–];

$\underline{E}_{imp}$ $= E_{imp}/1\,425$ [–];

E_{imp} $= 0{,}5\, m v_o^2$;

F_{bow} is maximum frontal collision force, in MN;

F_0 is reference collision force (= 210 MN);

E_{imp} is available impact energy, in MNm;

L_{pp} is length of the ship, in m;

m is mass of the vessel plus added mass upon movement in the longitudinal direction, in 10^6 kg;

v_0 is initial speed of the ship in m/s.

From the energy balance, the maximum actual crumble length u_{max} is determined using Formula (A.14):

$$u_{\text{max}} = \frac{\pi E_{\text{imp}}}{2 F_{\text{bow}}} \tag{A.14}$$

The associated impact duration, t_0, is represented by Formula (A.15):

$$t_0 \approx 1{,}67 \frac{u_{\text{max}}}{v_0} \tag{A.15}$$

In the absence of detailed analysis, example values of the forces due to impact with sea going ships given in A.3.4 may be used.

A.3.4 Examples of impact forces

Table A.11 gives some example values of the forces due to ship impact on inland waterways.

Table A.11 — Indicative values for the dynamic forces due to ship impact on inland waterways

CEMT class	Reference type of ship	Length l m	Mass m t	Force[a] F_{dx} kN	Force F_{dy} kN
I		30–50	200–400	2 000	1 000
II		50–60	400–650	3 000	1 500
III	“Gustav König”	60–80	650–1 000	4 000	2 000
IV	Class “Europe“	80–90	1 000–1 500	5 000	2 500
Va	Big ship	90–110	1 500–3 000	8 000	3 500
Vb	Tow + 2 barges	110–180	3 000–6 000	10 000	4 000
Vla	Tow + 2 barges	110–180	3 000–6 000	10 000	4 000
Vlb	Tow + 4 barges	110–190	6 000–12 000	14 000	5 000
Vlc	Tow + 6 barges	190–280	10 000–18 000	17 000	8 000

[a] The forces F_{dx} and F_{dy} include the effect of hydrodynamic mass and are based on background calculations, using expected conditions for every waterway class.

Table A.11 *(continued)*

CEMT class	Reference type of ship	Length l m	Mass m t	Force[a] F_{dx} kN	Force F_{dy} kN
VII	Tow + 9 barges	300	14 000–27 000	20 000	10 000

a The forces F_{dx} and F_{dy} include the effect of hydrodynamic mass and are based on background calculations, using expected conditions for every waterway class.

In harbour areas, the forces given in Table A.11 may be reduced by a factor of 2.

Table A.12 gives some example values for design forces due to ship impact for sea waterways.

In harbour areas, the forces given in Table A.12 may be reduced by a factor of 0,5.

For side and stern impact, it is recommended to multiply the forces given in Table A.12 by a factor of 0,3, mainly because of reduced velocities. Side impact may govern the design in narrow waters where head-on impact is not feasible.

Table A.12 — Indicative design values for the dynamic interaction forces due to ship impact for sea waterways

Class of ship	Length l m	Mass m[a] t	Force F_{dx}[bc] kN	Force F_{dy}[bc] kN
Small	50	3 000	30 000	15 000
Medium	100	10 000	80 000	40 000
Large	200	40 000	240 000	120 000
Very large	300	100 000	460 000	230 000

a The mass m in tons (1 t = 1 000 kg) includes the total mass of the vessel, including the ship structure, the cargo and the fuel. It is often referred to as the displacement tonnage. It does not include the added hydraulic mass.

b The forces given correspond to a velocity of about 5,0 m/s. They include the effects of added hydraulic mass.

c Where relevant, the effect of bulbs should be accounted for.

A.4 Impact from airplanes

A.4.1 Impact model

A.4.1.1 Accident model

The probability of a structure being hit by an airplane is very small. Only for exceptional structures like nuclear power plants, where the consequences of failure can be very large, is it mandatory to account for aircraft impact during design.

For air corridors, the probability of an impact force F_c exceeding a value X in a period T_{ref} is given by[9]:

$$P(F_c > X) = n T_{ref} \lambda A_b P_{na} P(F_c > X | \text{impact}) f_s(y) \quad \text{(A.16)}$$

where

n is the number of planes passing per time unit through an air corridor (traffic intensity);

T_{ref} is time period of interest (usually one year);

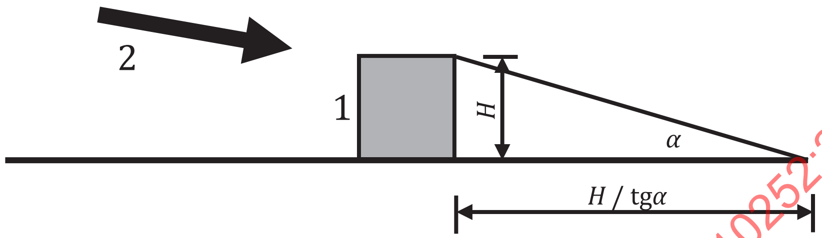
λ is probability of having a crash per unit distance of flying;

$f_s(y)$ is distribution of ground impact perpendicular to the corridor direction, given a crash;

A_b is the plan area of the building including the shadow area;

P_{na} is probability of not avoiding a collision with a specific object, given an airplane on collision course.

The area A_b is the area of the building itself, enlarged by a so-called shadow area (see Figure A.21). The strike angle α is random.



Key

1 building

2 flight direction

α strike angle

Figure A.21 — Strike area, A_b, for an airplane crash including shadow area

For the vicinity of an airport (at a distance r) the impact force distribution is based on Formulae (A.17) and (A.18)[9]:

$$P(F_c > X) = n\,T_{ref}\,P_{na}\,\Lambda(r)\,A_b\,P(F_c > X\,|\,\text{impact}) \tag{A.17}$$

$$\Lambda(r) = \frac{\bar{\Lambda} R}{2r} \tag{A.18}$$

where

$\bar{\Lambda}$ is average air plane collision rate for a circular area with radius R = 8 km;

$\Lambda(r)$ is collision rate for crash at distance r from the airport with $r < R$;

R is radius of airport influence circle;

r is distance to the airport.

Numerical values are presented in Table A.13.

Table A.13 — Numerical values for the air plane impact model

Symbol	Impact model	Numerical values
λ	Crash rate — military plane — civil plane	 10^{-8} km^{-1} 10^{-9} km^{-1}
$\bar{\Lambda}$	Average collision rate for airport area — small planes (<6 ton) — large planes (>6 ton)	 10^{-4} yr^{-1} km^{-2} 4 10^{-5} yr^{-1} km^{-2}
R	Radius of airport influence circle	8 km

Table A.13 *(continued)*

Symbol	Impact model	Numerical values
α	Strike angle	Mean 10^0 Standard deviation 10^0 Rayleigh distribution

A.4.1.2 Riera mechanical model for small planes

The impact force for airplanes is given as a force-time function or is directly obtained by finite element analysis using a detailed airplane model.

Riera[17] performed the initial work for the force-time function computation of the airplane impact. It is called the Riera model. The method has been used in many cases to date. The hit structure is assumed to be rigid; the crash is normal to the structure, and the aircraft can be simplified by a mass-spring system. The total reaction at the interface between the crashing aircraft and a rigid surface $F(t)$ is given by the Formula (A.19):

$$F(t) = F_c\left[x(t)\right] + \mu\left[x(t)\right]v(t)^2 \tag{A.19}$$

where

$x(t) = \int_0^t v(\xi)\,d\xi$	is the distance measured from the nose of the aircraft;
$F_c(x)$	is the dynamic force necessary to crush or deform the fuselage quasi-statically;
$\mu(x)$	is aircraft mass per unit length;
$v(x)$	is velocity of the uncrushed portion of the aircraft.

Both $F_c[x(t)]$ and $\mu[x(t)]$ are functions of the position along the aircraft, which is usually measured from the nose. Sugano, et al. (1993)[18] performed experiments consisting of an actual full-scale F-4 aircraft crashing into a massive concrete block, and included the mass distribution and crushing force for the F-4 aircraft, which are required input for the Riera method.

A.4.1.3 General mechanical model for large planes

The model of a fully rigid structure as assumed in the Riera model is not very realistic in most cases. A more realistic analysis for large planes, however, asks for a complex interaction between the aircraft structure and the building structure. An integrated non-linear dynamic FEM model may be used with all elements of the aircraft at $t = 0$ having a velocity v_0 and the structural elements being at rest. Elements of the aircraft as well as the building structure at the impact zone will be completely deformed and destroyed. Actually, combination with fire modelling also seems realistic.

A.4.2 Elaboration and examples

Figure A.22 shows an example of mass per unit length, $\mu[x(t)]$, for the F-4 Aircraft[19]. Figure A.23 shows an example of measured crush force, $P_c[x(t)]$, for the F-4 aircraft including skids and rockets[19].

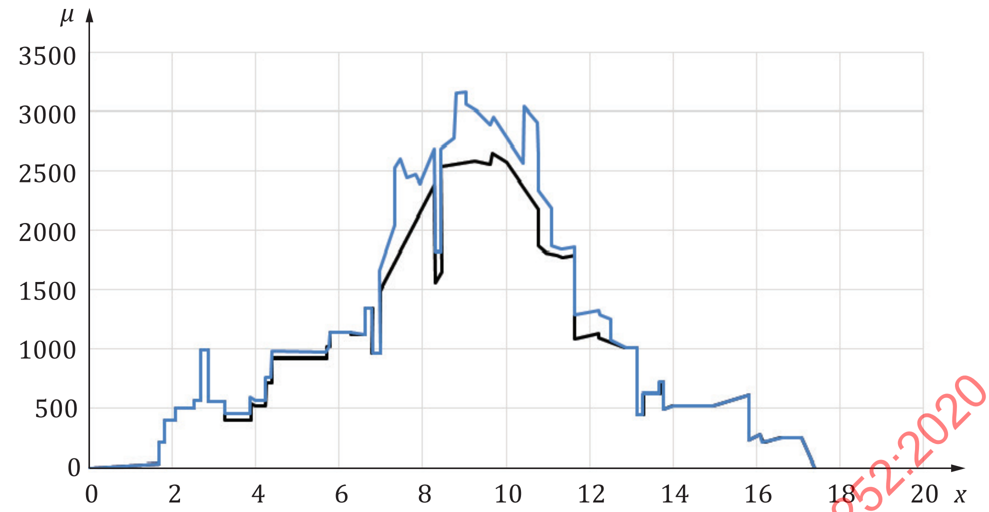


Figure A.22 — Example of mass per unit length, $\mu(x)$, for the F-4 Aircraft with (upper curve) and without (lower curve) skids and rockets[19]

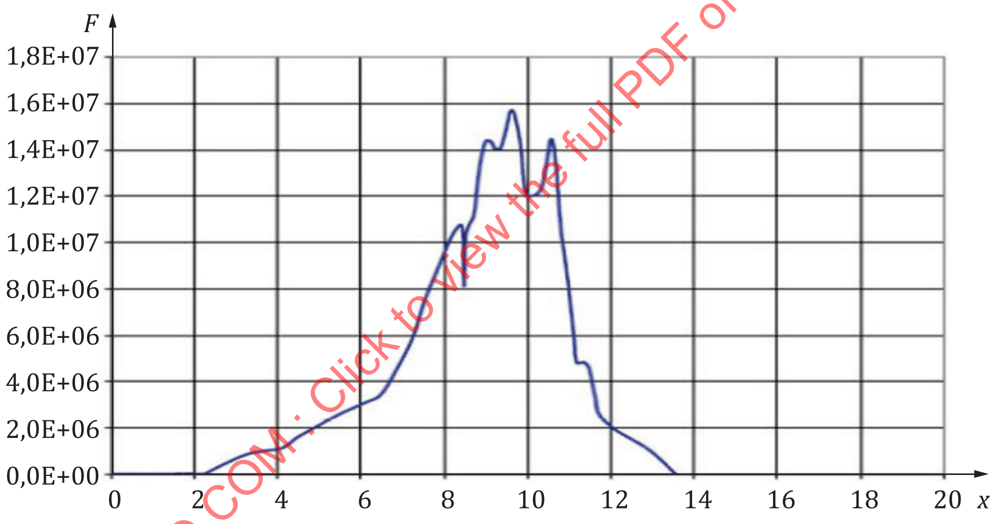


Figure A.23 — Example of measured crush force, $F_c(x)$, for the F-4 aircraft including skids and rockets[19]

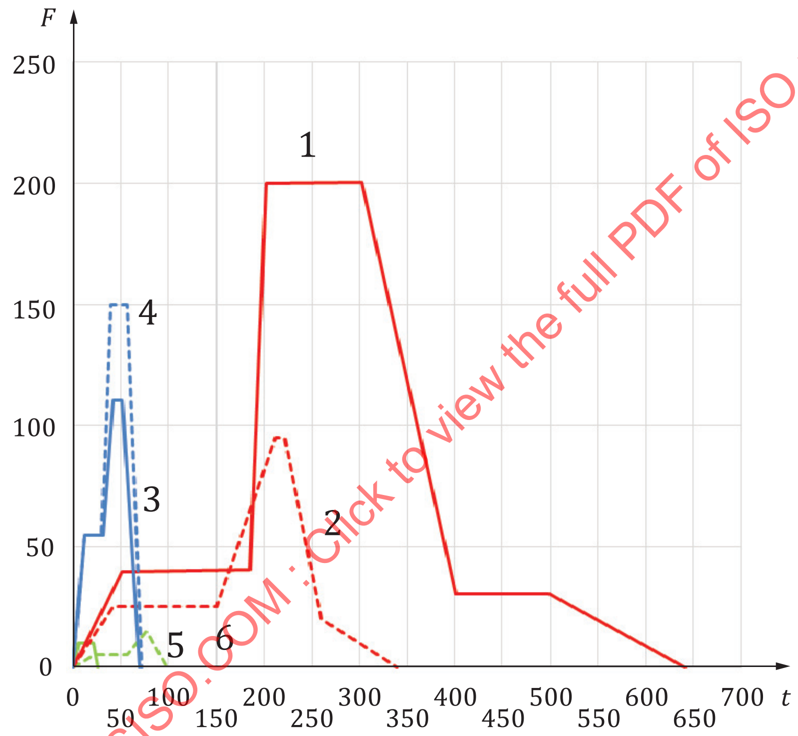
The collision speed and mass and stiffness distributions of the aircraft are important in evaluating the impact load caused by the aircraft collision. Any speed from among the takeoff and landing speed, gliding speed and cruising speed is generally used as the collision speed. In the case of large passenger aircraft, the speed at the time of take-off and landing is generally used because this is when accidents often occur. Table A.14 shows the aircraft mass and flight speeds for various types of aircraft. Figure A.24 presents the force-time functions for various aircraft types according to Riera.

Table A.14 — Aircraft weights and flight speeds for a typical fighter aircraft, a passenger aircraft, and a small aircraft[20]

Aircraft type	Typical model	Mass kg	Flight speed (horizontal) m/s
Fighter aircraft	F-15C	20 244	130[a]
	F-16C	11 372	150[a]
Passenger aircraft	B747-400	394 625	256[b]
Small aircraft	Cessna172	1 089	56[b]
Small rotorcraft	AS350B	1 900	65[b]

[a] Gliding speed.

[b] Cruising speed.



Key

F force (MN)

t time (ms)

1 commercial passenger aircraft (Boeing 747-400)

2 commercial passenger aircraft (Boeing 720)

3 military jet (Phantom RF-4E)

4 military jet [MRCA (Tornado)]

5 private passenger aircraft (CESSNA)

6 private passenger aircraft (LEAR JET)

Figure A.24 — Force-time functions for various aircraft types according to Riera[17]

The collision speed of the entire aircraft is often used as the collision velocity for the design when evaluating penetration and scabbing. This is performed by assuming the scattering of relatively hard components, such as the engine.

The maximum value of the force-time function or an equivalent static force with an impulse equivalent to that of the force-time function may be used in the design consideration because of the necessary superposition with the other loads.

Meanwhile, the impact experimental results are effective as reference data of the load calculation. Many simulation analyses for the experimental results were published in the literature (e.g. impact experiments using an aircraft engine[22][23], and also an entire airplane, for example, the F4-Phantom fighter aircraft[24]–[26]). The Nuclear Energy Agency (NEA) and the Committee on the Safety of Nuclear Installations (CSNI) jointly conducted the IRIS 2010 project called "Improving Robustness Assessment of Structures Impacted by Missiles"[27]. One of the experimental datum sources, which was known as the Meppen Test[28], was conducted in Germany in the 1980s. An overview of the simulation analysis of the experiment results by 28 participating research institutes is shown in Reference [27].

The impact load evaluation has been performed in recent years by the coupled analysis of the airplane and the structure[29]–[33]. More rational designs have also become possible with the improvement of computer performances.

A.5 Impact from helicopters

A.5.1 Impact model

Impact caused by the helicopter fall (bad landing) and collision on buildings is in particular relevant for buildings with a heliport on the rooftop. According to the AIJ design guideline[10], the collision load acting on the building depends on the helicopter's mass, fall/descending speed at the time of collision, fuselage structural strength and stiffness, etc.

The collision load can be determined using one of the following methods:

— Approximation as a triangular wave using an energy-based approach; in some cases, the maximum value of the triangular wave may be used as a static design load.

— Determination of the design loads by applying a detailed analytical model.

The probability of a structure being hit by a helicopter is very small. For air corridors, referring to a case of an airplane, the probability can be calculated by Formula (A.16).

The traffic intensity and the helicopter failure intensity should be estimated on the basis of national or regional statistics, taking care of local circumstances like curvatures and grades.

A.5.2 Practical guidelines

A.5.2.1 Building factors that affect the impact risk from helicopters

To assess the impact risk resulting from helicopter fall and collision with building rooftop, the following factors should be taken into consideration:

— buildings with heliport on the rooftop,

— determination of the size and mass of the helicopter based on the flight condition of the helicopter,

— determination of a collision speed and collision target members of the helicopter based on the flight conditions,

— consideration of impacts caused by the helicopter fall and collision to the building columns, beams, and walls of the upper floors, though the main collision target is the slab of the rooftop,

— whether anti-collision measures such as fences and shock absorbers are present; anti-collision measures make it possible to reduce the design load in accordance with the effect of preventive measures.

A.5.2.2 Calculation of impact load of the helicopter

To calculate the collision load acting on the building, one should:

— take into account the collision parameters, such as peripheral location of the target buildings and mass and collision speed of the helicopter,

— assume that most of the kinetic energy at the time of collision is absorbed due to deformation of the helicopter,

— assume a collision with building rooftop to occur due to a helicopter free-fall from the sky,

— consider the helicopter's height, width, length, and position relative to the building when setting the active area of the collision load,

— assign conservative characteristic values to the physical constants of the structural and non-structural members of the collision body and the building. The strain rate effect selected should be in the appropriate range.

A.5.2.3 Design load specifications

Design values for accidental actions due to impact from helicopters may be given as an equivalent static force defined as follows[1]:

$$F_d = C\sqrt{m} \qquad \text{(A.20)}$$

where

C is 3 (kN kg$^{-0,5}$);

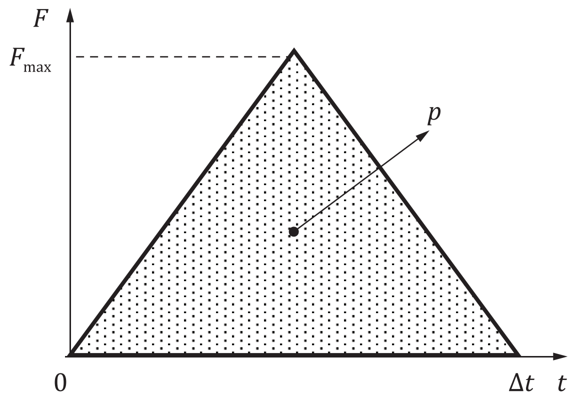
m is the mass of the helicopter, in kg.

The typical mass of a helicopter is several tons so the design force (kN) is in the order of 10^3 kN. The anticipated effective area of the impact force is within 7 m away from the edge of the landing pad, and the impact area is taken as 2 m × 2 m.

A.5.3 Example of impact load calculation

In this example, the impact load is estimated by assuming impulse during impact (time integral value of the impact load) to be equal to the momentum of the impact phenomenon (helicopter mass × impact velocity). Furthermore, duration time is obtained by dividing the crumbling height of the helicopter by the collision speed under the assumption that the collision speed is constant. Thus, the maximum load value is determined by the impulse and the duration time. For the dropped helicopter, the full height (of the helicopter) is equal to the cabin height, assuming a forced landing from the base of the helicopter.

The examples of the collision load are determined by using a method where the mass, cabin height and collision speed of the helicopter are assumed as shown in Table A.15 and Figure A.25. A triangular pulse is assumed here, but a rectangular pulse or another appropriate shape can be assumed according to the collision situation. Moreover, the acting position and area of the collision load are set up in consideration of the helicopter's height, width, and relative position of the building to the helicopter.



Key

F force (kN)

t time (s)

F_{max} maximum force (kN)

p impulse (mass m times velocity v) (kN s)l

Δt force duration (crumble length/impact velocity) (s)

Figure A.25 — Time history of impact force for impact by helicopters and fork lift trucks

Table A.15 — Parameters of helicopter and impact load

Mass m t	**Cabin height** H_0 m	**Crumbling zone of cabin height** u_o m	**Dropped height** h m	**Impact vel.** v km/h	**Force duration** $\Delta t = u_o/v$ s	**Maximum force** $F_{max} = 2mv/\Delta t$ kN
2	1,5	0,75	5	35,6	0,152	522
			10	50,4	0,107	1 046
4			5	35,6	0,152	1 046
			10	50,4	0,107	2 090

NOTE The crumbling zone u_o of cabin height is assumed to be 50 % of the cabin height H_0.

A.6 Impact from forklift trucks

A.6.1 Impact model

Forklift trucks are mainly present in large storage buildings and for a serious hazard for the storage racks and the building components. In some cases, the two systems are integrated.

The collision loads depend on the forklift's mass (inclusive its loading) and the collision speed.

The collision load can be determined using one of the following methods:

— Approximate the collision load as a triangular wave using an energy-based approach and apply the approximated maximum value of the triangular wave as a static design load.

— Determine the design loads by applying a detailed analytical model.

The impact force should be determined taking into account the local characteristics of the place. The object may be a wall or a column of the building or for instance any protective structure.

A.6.2 Practical guidance

A.6.2.1 Building factors that affect the impact risk from forklift trucks

To assess the impact risk resulting from forklift collision, the following factors should be taken into consideration:

— collision and rush of the forklifts due to erroneous operation for the buildings where forklifts work;

— not only the structural members of the building such as columns and walls, but also the non-structural members, such as the racks, the windows and entrance doors;

— the size and mass of the forklift based on the working condition of the forklift;

— whether anti-collision measures such as steps, bollards and grooves are present. If they are, it is possible to reduce the design load in accordance with the effect of the preventive measures.

A.6.2.2 Calculation of impact load of the forklift

To calculate the collision load acting on the building, it is important to take into account the collision parameters, such as peripheral location of the target buildings, working environment of the forklift, weight (sum of the self-weight of the forklift and the load weight) and collision speed of the forklift:

— assume a hard impact for which all kinetic energy at the time of collision is absorbed due to the deformation of the forklift;

— consider the forklift's height, width, length and position relative to the building when setting the active area and position of the collision load; and

— assign conservative characteristic values to the physical constants of the structural and non-structural members of the collision body and the building. The strain rate effect chosen should be in the appropriate range.

A.6.2.3 Design load specifications in Eurocode[1]

In the Eurocode, design values for accidental actions due to impact from forklift trucks are determined considering the dynamic behaviour of the forklift truck and the structure. As an alternative to a dynamic analysis, an equivalent static design force F may be applied.

It is recommended that the value of F be determined according to advanced impact design for soft impact. Alternatively, it is recommended that F be taken as $5W$ for vertical direction, where W(kN) is the sum of the net weight and hoisting load of a loaded truck. The height of the loading is determined corresponding to the conditions of the anticipated event and the basic height is 0,75 m above the floor. Horizontal loads due to acceleration or deceleration of forklifts may be taken as 30 % of the vertical axle loads W(kN).

A.6.3 Example of impact load calculation

In this example, the impact load is estimated by assuming impulse during impact (time integral value of the impact load) to be equal to the momentum of the impact phenomenon (forklift mass × impact velocity). Furthermore, duration time is obtained by dividing the crumbling length of the forklift by the collision speed under the assumption that the collision speed is constant. Thus, the maximum load value is determined by the impulse and the duration time.

Some examples of the collision load determined by using a method where the mass, crumbling length and collision speed of the forklift are assumed as shown in Table A.16 and Figure A.25. The acting position and area of the collision load are set up in consideration of the forklift's height, width and the relative position of the building to the forklift.

Table A.16 — Parameters of forklift and impact load

Total mass	Length	Crumbling length	Impact vel.	Force duration	Maximum force
m	l_0	u_o	v	$\Delta t = u_o / v$	$F = 2mv/\Delta t$
t	m	m	km/h	s	kN
3	3,0	0,30	10	0,11	150
			15	0,072	350
9	4,8	0,48	10	0,17	290
			15	0,12	650
28	7,3	0,73	10	0,26	590
			20	0,13	2 400

NOTE The crumbling length u_o is assumed to be 10 % of the total length l_0.

The forces in Table A.16 are based on the assumption that the truck absorbs all the energy; in many cases; however, it is more likely that the truck is relatedly stiff and the slender structural element will absorb all energy. In that case, the element can may be deformed in such a way that the resistance to carry the regular load is strongly reduced and not sufficient anymore. The global structural behaviour then depends on the capacity of the structure to find a second load path.

A.7 Impact by falldown streaming and sliding materials

A.7.1 Free falling objects

There are occasions when designers are concerned about objects dropping on structures. Examples include crane operations when lifted loads are handled above structures, during renovations when elements accidentally are not fully fixed. The falling material can also have natural causes.

The analysis often involves at least two considerations:

— penetration of roofs impacted by debris, as by punching shear, and

— failure of structural elements by flexure and end shear.

Lightweight roofs, such as those of metal deck, are unlikely to experience punching failures. Usually, their mass is small enough and their susceptibility to local bucking failures forces them to respond as flexural beam or plate elements. Theoretically, it is possible for high-velocity debris to punch through more massive concrete decks, which generally have brittle shear characteristics. However, the velocity of objects typically under consideration, falling from a few tens of meters height, usually is not high enough to cause punching failures (see Reference [34] for methods to calculate punch potential for high velocity missiles). Nevertheless, rigid objects falling on concrete decks likely cause local penetration and spalling.

The solution of the second consideration — the potential for flexural and end shear failures — is achieved through the assessment of the dissipation of the energy and momentum of the falling object. An object in freefall has the following energy and momentum:

$$E = 0{,}5\, m_{\mathrm{f}}\, v_{\mathrm{r}}^{2} = m_{\mathrm{f}}\, g\, h \qquad \text{(A.21)}$$

$$p = m_{\mathrm{f}}\, v_{\mathrm{r}} = m_{\mathrm{f}} \sqrt{2gh} \qquad \text{(A.22)}$$

where

E is the kinetic energy of the falling object;

p is the momentum of the falling object;

m_f is the mass of the falling object;

h is the height of fall;

g is the acceleration due to gravity.

Normally, one would not assume that the impact is elastic (i.e. the falling object does not bounce upon striking the structure). Usually, it is assumed that the impact is plastic, and the response depends on whether the falling object remains intact or disintegrates upon impact. If one assumes that the falling object (e.g. a piece of rigid machinery dropped by a crane) remains intact, then a straightforward approach to evaluating the response is to impart the momentum of the falling object to a mass represented by the combined mass of the falling object and the engaged structure, and determine the resulting kinetic energy that is dissipated by the structure, as per Formula (A.23):

$$E_\mathrm{c} = \frac{{m_f}^2}{m_f + m_s} gh \qquad \text{(A.23)}$$

where

E_c is the kinetic energy of the combined mass;

m_s is the mass of the engaged structure.

While this approach accounts in some measure for dissipation of energy due to the collision through deformation of the impacted surfaces, generation of noise and heat, etc., often it is conservative, because it assumes impact is instantaneous whereas the force between the falling object and structure has a finite duration, and is on the order of the time it takes for the deflecting structure to reach its elastic limit. Moreover, the falling object rebounds and certain kinetic energy can be stored in the falling object. Also complicating the analysis is the estimation of m_s. because not all of the mass of the structure is engaged to move at the same velocity. Hence, an equivalent mass which is calculated by the deformation mode of structure can be assumed.

Assuming that the dropped mass disintegrates (e.g. a falling section of unreinforced masonry that breaks apart when striking a roof), then it can be assumed that the momentum of the falling mass is transferred to the engaged structure, with the pieces of the broken falling mass dispersed laterally in all directions across the roof structure. In this case, the kinetic energy that is dissipated by the structure can be taken as:

$$\mathrm{E}_c = \frac{{m_f}^2}{m_s} gh \qquad \text{(A.24)}$$

This approach also is conservative in most cases. For scenarios in which the falling object is assumed to disintegrate, the neglected rise time for the interacting force is on the order of the time it takes for the falling object to break into pieces. As an alternative to both of the assumptions presented above, it can be assumed that all the kinetic energy of the falling body shall be dissipated through deflection of the engaged structure. Using these energy balance approaches, the problem becomes one of determining whether the structure should remain elastic and, if not, how much ductility and nonlinear response is necessary to dissipate the kinetic energy and bring the moving portions of the structure to rest before failure. This can be done by tracking the initial kinetic energy plus the change in potential energy as the structure deforms after the impact, and plotting it against the energy dissipated through conversion to strain energy, first through elastic response and then through post-elastic response, of the structure.

The structure comes to rest, and survives, if the energy dissipated at any particular deformation exceeds the imparted kinetic energy plus the added change in potential energy before a structural resistance, such as ultimate flexural capacity or end shear capacity, is exceeded. Of course, more rigorous approaches are possible. Sophisticated software is suited for such problems. The problem is complicated by the possibilities about where the impact can occur: will the object strike the structure near midspan of an element, inducing primarily flexural response, or will it strike near a support point, thereby rapidly raising end shear that often produces a relatively brittle failure mode? This is often addressed by postulating several scenarios and testing the outcomes, ultimately considering the probabilities of certain impact locations while making decisions. For instance, one can consider how far horizontally a section of masonry, if it becomes dislodged, can land from the face of a building under renovation. Also, one can control the swing of a crane hoisting an object over a building if there are particularly vulnerable impact locations that should be avoided. One can assess the probability of failure due to impact at a random location by calculating the roof area over which impact is equally likely and assessing the percentage of that roof area over which impact will likely cause failure.

Finally, the force on the structure as well as the duration may be estimated from using the formulae from Clause 6.

A.7.2 Falling or sliding of geo-material

There are several kinds of impact phenomena due to falling or sliding of geo-material such as a rockfall and debris flow. These phenomena are usually caused by excessive land utilization or lack of consideration regarding extreme rainfall. In general, impact phenomenon by geo-material is assumed as a collision of two solid bodies and standard impact force F_{Hertz} is calculated by the Hertz's contact theory[35][36]. After that, the real impact force is usually revised by certain experimental correction factor as follows.

$$F_{\mathrm{design}} = \beta \cdot F_{\mathrm{Hertz}} \tag{A.25}$$

where β is an experimental correction factor. See also A.7.4.

A.7.3 Rockfall

A.7.3.1 General

A rockfall is defined as a fragment of rock (a block) detached by sliding or falling, that falls along a vertical or sub-vertical cliff, descending the slope by bouncing and flying along ballistic trajectories to the valley floor. An accident caused by a falling rock is common problem for mountainous regions in the world. In the case of direct collision, instantaneous huge impact load is arisen by rockfall. To avoid this direct collision and soften impact action, cushion material such as sand or rubber is usually used on the structures. For example, when a falling rock collides with sand and penetrates into sand layer and finally distributed pressure acts on the surface of the structure. Thus, many full-scale experiments have been performed in order to determine reliable design force of a falling rock with sand cushion.

A.7.3.2 Impact action of a rockfall

Collision of a rockfall with sand cushion is an energy dissipation phenomenon with plastic deformation of sand layer. However, for the sake of convenience, the maximum impact force equation of rockfall is based on the improved Hertz's contact theory and each parameter is empirically determined by full scale impact experiments. See also References [35] and [36].

$$F_{\max} = 2{,}108(mg)^{2/3}\,\lambda^{2/5} H^{3/5} \tag{A.26}$$

where

F_{max} is maximum impact force, in kN;

m is mass of rock, in t;

g is acceleration of gravity, in m/s^2;

λ is Lamé's constant;

H is equivalent drop height of rock, in m.

In Formula (A.26), Lamé's constant λ is regarded as a control variable and adequate value is determined by regression of full-scale experimental results. The value of λ (kN/m^2) is usually used in the range 1 000 to 10 000, depending on the state of the sand cushion.

On the other hand, the Formula based on the conservation of momentum has also been presented:

$$F_{max} = 18,6\, \beta_r m \frac{\sqrt{2gH}}{t_d} \tag{A.27}$$

where

t_d is duration of impact force, in s
t_d = (0,048 1 + 0,000 64H) m^{0,27} C_c;(usually between 5 ms and 40 ms);

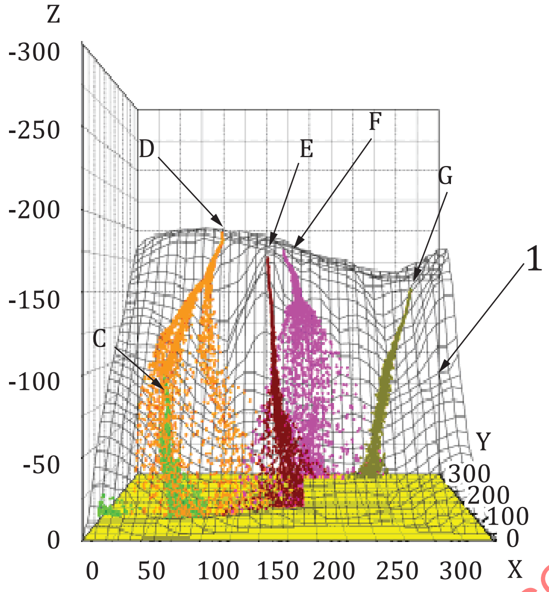
C_c is uniformity coefficient of cushion;

β_r is correction factor concerning cushion
β_r = −5,34 d + 5,84 for d < 0,9
β_r = 1,03 for $d \geq 0,9$;

d is thickness of cushion (m).

To decide design condition such as mass and equivalent height of rock, rockfall trajectory simulations are usually performed. In the simulation, a rock is assumed to be a rigid body. The motions such as jump, collision, slide or rotation of a rock are considered and reaction force by slope is calculated using a numerical time integration scheme]. Figure A.26 shows an example of trajectory simulation from five different release positions C to G determined by real slope condition.

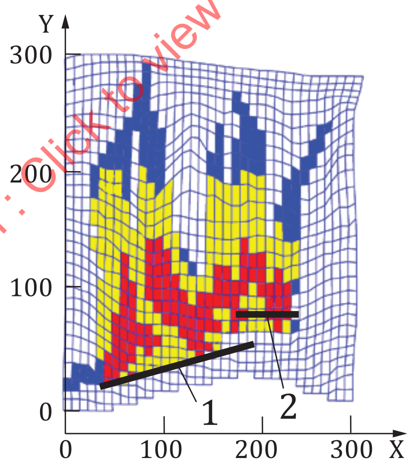
Figure A.27 shows the hazard map of rockfalls based on the kinetic energy of rockfall using trajectory simulation results. Relatively large energies (over 3 000 kJ) are observed at lower slope and even extremely large (more than 6 000 kJ) area is also found. The use of this kind of simulation method is more effective and rational than the use of equivalent coefficients of friction to estimate the risk of rockfall accident for the slope of the complex topography.



Key

X, Y and Z	direction axis (m)
1	mountain slope used for simulation
C, *D*, *E*, *F* and *G*	rockfall sources

Figure A.26 — Example of trajectories of rockfall[52]



Key

X, Y	direction axis (m)
1	embankment
2	barrier
(blue)	$0 \leq E_r < 3\ 000$
(yellow)	$3\ 000 \leq E_r < 6\ 000$
(red)	$6\ 000 \leq E_r$
E_r	kinetic energy of rockfall (kJ)

Figure A.27 — Example of hazard map of rockfall[52]

NOTE The forces given in this subclause do not include dynamic effects inside the structure.

A.7.4 Debris flows

A.7.4.1 General

Debris flows (not mud flows) are geological phenomena in which water-laden masses of soil and fragmented boulder rush down mountainsides and run into valley. They generally descend steep channels and their average speed surpasses 10 m/s (more than 20 miles per hour). The volume of debris flows is larger than 100 000 m^3 frequently in mountainous regions worldwide such as the Alps, United States, Japan, Indonesia and South American countries. A design impact load and/or energy of boulders contained in a debris flow on check dam structures, are described in this annex. The check dam structures discussed herein are usually constructed in rivers in mountainous area. In the descending process, huge boulders and gravels gather in the front part of debris flow due to segregation mechanisms and significantly increase collision energy of the huge boulders. To prevent consequences on human lives and society, check dam structures are usually constructed in the upstream site and are expected to catch the debris flow and boulders directly, which mitigates energy of the debris flow in the downstream site[64].

A.7.4.2 Mass and velocity of boulder used in the design

The impact energy and impact load for the collision between the boulder and check dam are generally determined with a mass and velocity of the boulders. It is implicitly assumed in the structural design that the boulder is spherical and collides with the check dam perpendicularly.

A diameter of the boulder is determined as a probabilistic value S which is the event probability for the design debris flow from the upstream site to the check dam. For example, S can be determined as 95 % non-exceedance value of boulders size of samples. In this case, the tenth diameter in 200 boulders investigated in upstream of the river is determined as the design boulder diameter.

The velocity v is associated with the peak discharge calculated by using a probabilistic precipitation, for instance a return period of 100 years[65] to [66].

A.7.4.3 Impact action on the concrete check dam

The force F acting on the concrete structures by boulder collision is given by Formulae (A.28) to (A.32), which are based on the Hertz's contact theory and observation result in the site[67].

$$F=\beta \cdot n\alpha^{3/2} \tag{A.28}$$

$$n=\sqrt{\frac{16R}{9\pi^2\left(K_1+K_2\right)^2}} \tag{A.29}$$

$$K_1=\frac{1-{v_1}^2}{\pi E_1}, \qquad K_2=\frac{1-{v_2}^2}{\pi E_2} \tag{A.30}$$

$$\alpha=\left(\frac{5v^2}{4n_1 n}\right)^{2/5}, \qquad n_1=\frac{1}{m_2} \tag{A.31}$$

$$\beta=(E+1)^{-0{,}8}, \qquad E=\frac{m_2}{m_1}v^2 \tag{A.32}$$

where

R is the radius of boulder, in m;

E_1 is the average Young's modulus of concrete during failure process (modulus of deformation for ultimate strength), in N/m^2;

E_2 is the Young's modulus of boulder material, in N/m^2;

ν_1 and ν_2 are the Poisson's ratios of concrete and boulders material, respectively;

m_1 and m_2 are the masses of concrete and boulders, respectively, in kg;

β is the coefficient determined by experiments.

v is the velocity of the boulders

The average Young's modulus of concrete E_1 is usually set as 1/10 of the Young's modulus of concrete in the elastic range.

A.7.4.4 Log jams in rivers

A different type of debris related accidental event is the development of log jams in rivers that can lead to the rise of water levels and subsequent flooding as well as increased hydraulic loads (see Reference [68]).

Annex B
(informative)

Guidance on detailed explosion analysis

B.1 Internal gas and high energy explosions in buildings

B.1.1 Models for explosion

B.1.1.1 General

Three different categories of models for prediction of overpressure due to internal explosions are discussed. These are empirical and codified models, CFD-models and phenomenological models.

B.1.1.2 Empirical and codified models

Numerous empirical methods predicting explosion overpressures based on explosion venting are published in the literature. The models are valid for a limited range of variables such as volume, burning velocity, mass of fuel (air mixture) and vent areas. The empirical correlations are based on the concept of a vent coefficient K, as per Formula (B.1):

$$K = \frac{A_s}{A_v} \tag{B.1}$$

where

A_s is the area of the side of the enclosure;

A_v is the area of the vent opening.

Venting panels should open at a lower pressure than that can be sustained by the surrounding structure and should be as light as possible. The vents should be designed such that they open at a pressure less than or equal to half of the (desired) design overpressure $p_{red} = p_d$.

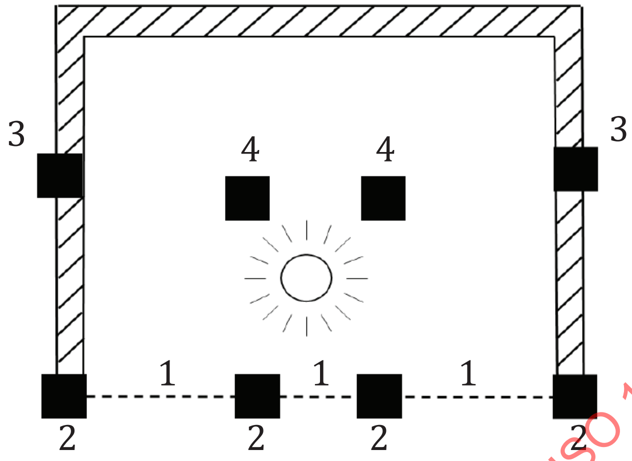
In determining the capacity of a venting panel, account shall be taken of the dimensioning and construction of the supporting frame of the panel.

Where a vent is used, after the first positive phase of an explosion with an overpressure, a second phase can follow with an under-pressure. This effect should be considered in the design where relevant.

Where used, venting panels should be placed close to the possible ignition sources, if known, or where pressures are expected to be higher. They should be discharged at a suitable location that will not endanger personnel or ignite other material. The venting panel should be restrained so that it does not become a missile in the event of an explosion. The design should limit the possibilities that the effects of the fire cause any impairment of the surroundings or initiates an explosion in an adjacent room.

A venting solution shall not be used if toxic dust, or other associated toxic substances, which cannot be vented to the atmosphere are present unless allowed for by an appropriate risk assessment.

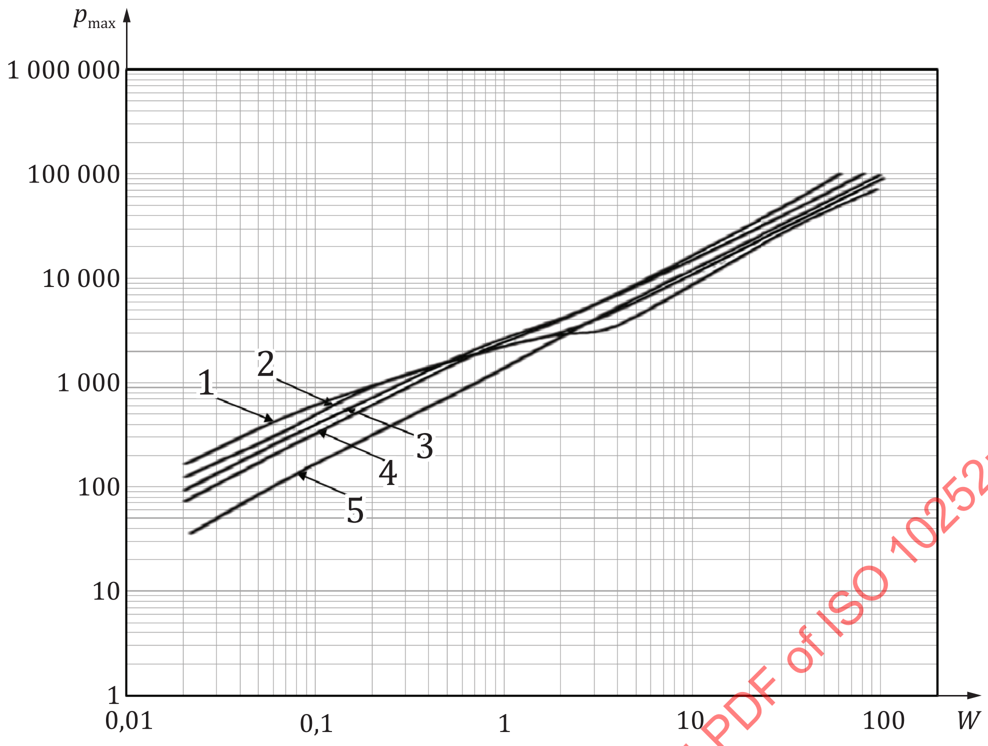
Loads of structural members are not only determined by the peak pressure in the room but also depend on the total configuration. For instance, ignition of a vapour inside a building releases heat that generates overpressure in the confined space. That overpressure drives flow toward vent openings. The loads to which structural elements are subjected can be a function of the position of those elements relative to shock fronts (particularly for high-energy explosions) and heat-induced flow. For example, column 4 in Figure B.1 can be loaded relatively uniformly on all sides for a vapour explosion and can survive the explosion, whereas column 2 within the venting path can experience unequal loads on opposing faces, more significantly threatening its survival. Of course, these are generalizations that express concepts rather than provide specific guidance.



NOTE Panels 1 are venting panels; 2, 3 and 4 are columns in different loading situations

Figure B.1 — Floor plan with differently situated columns

Interior detonations of high-energy explosives can be addressed conservatively by calculating the peak gas overpressure, p_{max} from Figures B.2 and B.3 for the particular explosive compound. The interior surface of interest shall be designed for this gas overpressure applied statically.



Key

p_{max} peak gas pressure (kPa)

W loading density (kg/m^3)

1 = HBX-3

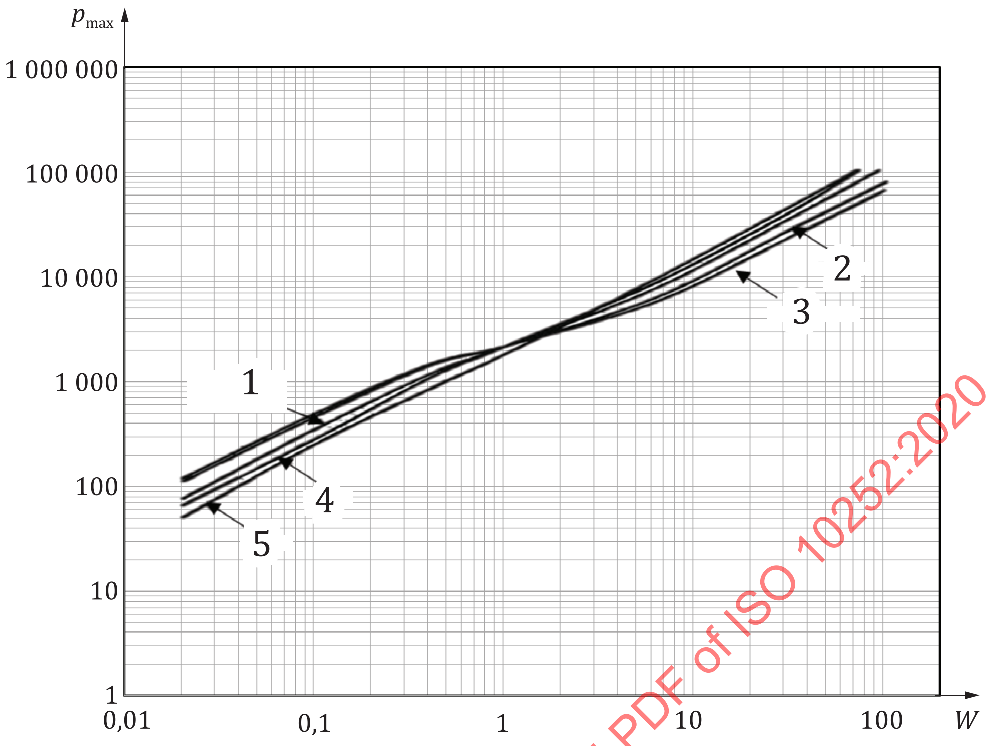
2 = TNT

3 = C4

4 = RDX

5 = ANFO

Figure B.2 — Peak gas overpressure for HBX-3, TNT, C4, RDX, and ANFO (ASCE/SEI 59-11)



Key

p_{max} peak gas pressure (kPa)
W loading density (kg/m^3)
1 = pentolite (50/50)
2 = HBX-1
3 = tritonal
4 = HMX
5 = PETN

Figure B.3 — Peak gas overpressure for HMX, PETN, PENTOLITE, HBX-1, and TRITONAL (ASCE/SEI 59-11)

To qualify for this approach the following has to apply:

a) The blast occurs internal to the structure.

b) The structure has no unusual geometric irregularities in spatial form and has a maximum aspect ratio in plan dimensions less than 1,3.

c) The volume used to calculate the loading density is the “free” volume, which is the total volume minus the volume of all interior equipment, structural elements, etc.

d) The minimum covered vent area, A_v, is greater than or equal to 0,20 $V_f^{\ 2/3}$, where V_f is the internal free volume of the structure.

e) The unit mass of the vent(s) is less than or equal to 120 kg/m^2 (25 psf).

This procedure conservatively applies the gas overpressure as a static load on the affected surfaces. In some situations, this results in an acceptable design. In others, the design can be excessively conservative due to venting and the comprehensive methods of Reference [69] [UFC 3-340-02 (DoD 2008)] can be used. See Reference [70] for more information.

As an alternative to the approach described above, one might follow rigorous procedures that model the explosive environment and the structural response accurately.

B.1.1.3 CFD-models

In the computational fluid dynamics (CFD) approach, the physics are resolved numerically by dividing space into small control volumes and implementing models for various phenomena like fluid flow and turbulence. In each cell, all variables are assumed constant in one-time step, and based on the flow balance and fluxes, as well as physics taking place inside the cell in the next time step, the variables may change. For explosions, further models have to be incorporated compared to a standard CFD-model, as flame propagation and combustion have to be modelled. Thus formulae for:

— mass balance (continuity),

— impulses,

— enthalpy,

— turbulence,

— fuel transport and mixture fraction

are solved for each time step and control volume.

If only blast pressures in the far field are to be assessed, models like Reference [71] may be used. This is a reduced model. Based on a simplified geometry representation, and assumption on constant (high) burning velocity, blast curves for a specific situation can be generated. By treating the geometry simplified, details about the flame development are lost, and an explosion strength has to be assumed.

Several of the weaknesses with the different empirical curves are avoided with this approach, but on the other hand, the simulation time is significant and nearly the same as for a full CFD-simulation. Also, advanced CFD-models can be used for such a simplified approach, if it is considered too expensive to generate the detailed geometry model.

There is a range of CFD-models that claim to simulate gas explosions as well as high energy explosions. To be able to develop such tools properly, good knowledge about experiments and physics is needed.

Special purpose CFD-models have a greater potential to perform well, as all of these simulators are developed by people doing experimental and theoretical work within gas explosions. Still, significant differences are seen between the models, both with regard to applicability and validity.

General purpose structural programs with CFD-packages can analyse the blast environment and structural response.

Even with refined analysis grids (typically 0,5 m to 1,0 m in an offshore or industrial module), it is difficult to capture the explosion physics taking place at small scales. Examples of such sub-grid models are:

a) turbulence,

b) flame propagation/wrinkling,

c) water deluge.

Since geometry and scenario details are of high importance for explosions, the special purpose CFD-models need to represent geometry properly. Porosities/blockages are calculated due to geometry mapping onto the simulation grid. Geometries can be either defined by hand or imported from CAD systems.

The output from a simulation with CFD-models has few limitations. Output can be either pressures, flames or any other parameter modelled, either as a 2D/3D field plot of one variable at one or more time steps or transient pressure traces at certain locations or wall panels.

In practice, the accuracy of the CFD models is limited by:

— available computation power limiting the numerical resolution,

— the accuracy of numerical models,

— the underlying empirical sub-models for reaction zone, turbulence generation, turbulence length scale, turbulent combustion, etc.

B.1.1.4 Phenomenological models

For the prediction of vapour explosions inside vented compartments, there is one group of models referred to as phenomenological models as implemented several computer programs. These are based on 1D considerations, trying to model some of the physics involved in the process. A typical module may be divided into a limited number, typically less than 10 individual control volumes. Effects not picked up by the physics are handled by a range of empirical constants found from calibrating against a range of relevant experiments.

Input is a rough geometry model. The available vent area is of importance, and for each subdivision of the module in the simulation, the blockage has to be estimated. The chamber of ignition has to be given. Now and then, these models are applied to other situations than developed for, e.g. less confined modules that are not typical 1D situations. High uncertainty should be expected for such applications. In principle, such models can be considered just as CFD-models with a very coarse (poor) grid resolution.

The output from phenomenological models is also limited, as the geometry may only be divided into a limited number of control volumes, thus the computed pressures are the average over a relatively large volume. No local pressure peaks are picked up; this is another reason why these models are of low value when applied to more open process areas and modules. Validation of these models is generally through comparison of simulation results with experiments.

B.1.1.5 Summary of empirical and codified models

Some of the relevant conclusions regarding a selection of prediction tools can be summarised as follows: CFD and phenomenological methods generally give fairly good accuracy (within a factor of two) so these models yield solutions that are approximately correct.

The limitations associated with empirical and phenomenological methods (simplified physics and relatively crude representation of geometry) can only be overcome through additional calibration.

It is recommended that "advanced" CFD codes allowing fully realistic combustion models and resolution of all obstacles be developed. However, it is likely to be many years before such tools are available readily. This is primarily due to the large computational expense of this approach.

Further evaluation of the methods can be found e.g. in References [72] and [73].

B.1.2 Stochastic modelling

B.1.2.1 Occurrence

Given that an explosion is an accidental action, the most important statistical description is the probability that the associated loading occurs. As a function of time, the occurrence of an explosion can be considered as a Poisson process:

$$P\left(\text{at least one explosion during } T\right) = 1 - \exp\left(-\lambda T\right) \approx \lambda T \tag{B.2}$$

where

λ is the probability of an explosion event per unit of time;

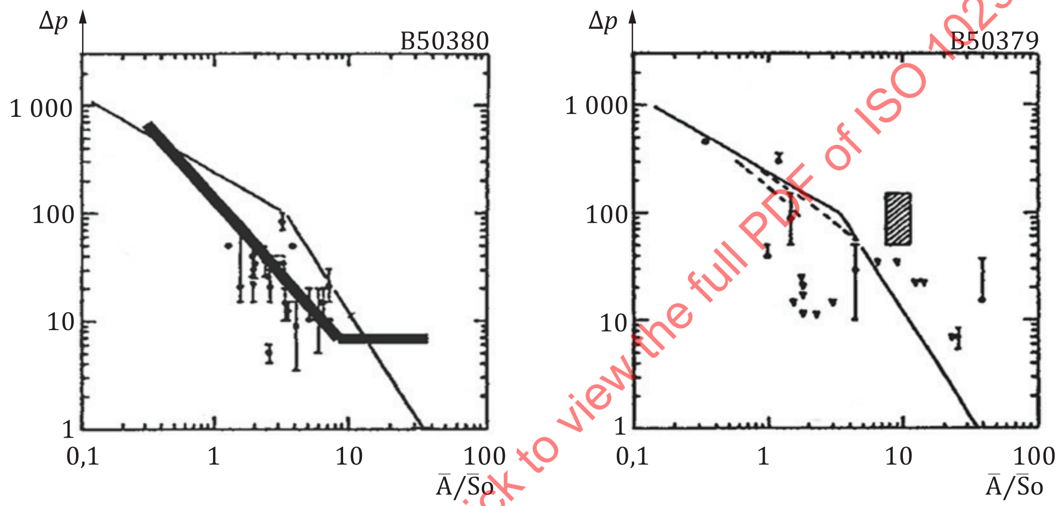
T is the considered (reference) period of time.

The value of λ depends on the type of explosion. For internal explosions in non-industrial buildings, an extensive survey performed in the UK gives some interesting data[74]. The values are in the order of

2 × 10^{-6} per year for dwellings and 6 to 14 × 10^{-6} per year for shops and industrial buildings. Present rates can be smaller due to improved safety measures. In the US, higher values (18 × 10^{-6}) have been found for residential buildings[75]. In the Netherlands, about 20 explosions are observed yearly, leading to $\lambda \cong 5 \times 10^{-6}$ per year as an average per dwelling/building. The occurrence rate, however, depends on whether small explosions (giving little damage only) are also taken into account.

B.1.2.2 Magnitude

The next step is to model the magnitude of the explosion, conditional upon occurrence. For internal explosions, the maximum pressure can be taken as the maximum of the pressure that causes vent coverings to release and the vent controlled pressure, which is the pressure that is achieved once venting has occurred. For the release pressure, a proper resistance model should be selected. The "vent control pressure" as observed in practice (as good as possible) can be estimated for vapour explosions from Figure B.4. The Eurocode recommendation (heavy line) may be considered as an average and the coefficient of variation is about 0,7.



Key

damage analysis

large scale experiment

safe recommendation

NOTE $\bar{A} = 0{,}64 A_v / A_s$ (with A_v = ventilation opening and A_s the total internal area); $\bar{S}_o = 7 S_o / c_o$ (dimensionless burning velocity), where S_o = 0,45 m/s for natural gas and c_o is 340 m/s. The thick line shows the Eurocode recommendation.

Figure B.4 — Relationship between the vent parameter $\bar{A} / \bar{S}_o$ and the peak pressure[76] for dwellings (left hand side) and industrial buildings (right hand side)

B.1.2.3 Random factors

Random factors affecting the CFD calculation and resulting internal pressures for a given explosion are:

— the position of leakage points,

— the flow rate of gas/liquid,

— air exchange rate due to ventilation,

— fuel concentration due to gas dispersion at one leak area,

— ignition events (position, time),

— flammability limits (in terms of fuel-air concentration),

— overpressure for a given homogeneous cloud made of a flammable fuel-air mixture.

For numerical values, see References [73] and [77].

B.2 Explosions in tunnels

B.2.1 General

Explosions in road tunnels can occur as a result of accidents with:

— LPG cars (tanks are usually small, order of 50 l),

— trucks carrying explosive materials, such as LPG,

— trucks carrying explosives (fireworks, ammunition).

In this clause, the scope is limited to trucks carrying LPG through road tunnels. The approach is based on Reference [78].

B.2.2 Probabilistic approach

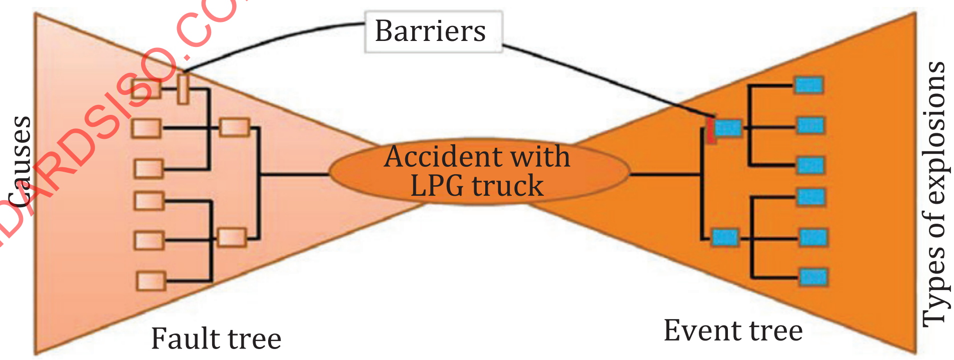
To determine the exceedance probabilities of explosions in tunnels, the following steps should be taken:

a) All scenarios possibly leading to an explosion should be identified.

b) The probability of occurrence of each of the scenarios should be determined or calculated.

c) For each type of explosion, the maximum pressure and the impulse should be calculated.

To determine the probability distribution of explosion in tunnels use is made of the bow tie model (Figure B.5), having a fault tree approach (backward logic) analyzing the crucial accident and an event tree (forward logic) to analyse the events following the accident.



NOTE A barrier refers to a possible safety device.

Figure B.5 — Bow tie model for explosion QRA

In the event tree, the consequences in terms of types of explosions are identified. To calculate the pressure and impulse of the explosion, it is essential to know the following details of the tunnel under consideration (based on traffic in one direction):

— Length of the tunnel, in m;

— Area of the cross section, in m^2;

— Wind speed and direction, in m/s;

Furthermore, the following (conservative) boundary conditions are defined:

— Location of the accident (distance *x* from entrance),

— Maximum size and contents of the tank of an LPG truck (60 m^3 tank with 50 m^3 gas),

— Time of ignition of a gas explosion: maximum length of an explosive gas cloud.

The pressure calculations can be done with full CFD calculations (so-called first principle analysis). An alternative is to use explosion models that have been specially developed for explosions in elongated volumes like tunnels[79] to [81].

B.2.3 Example

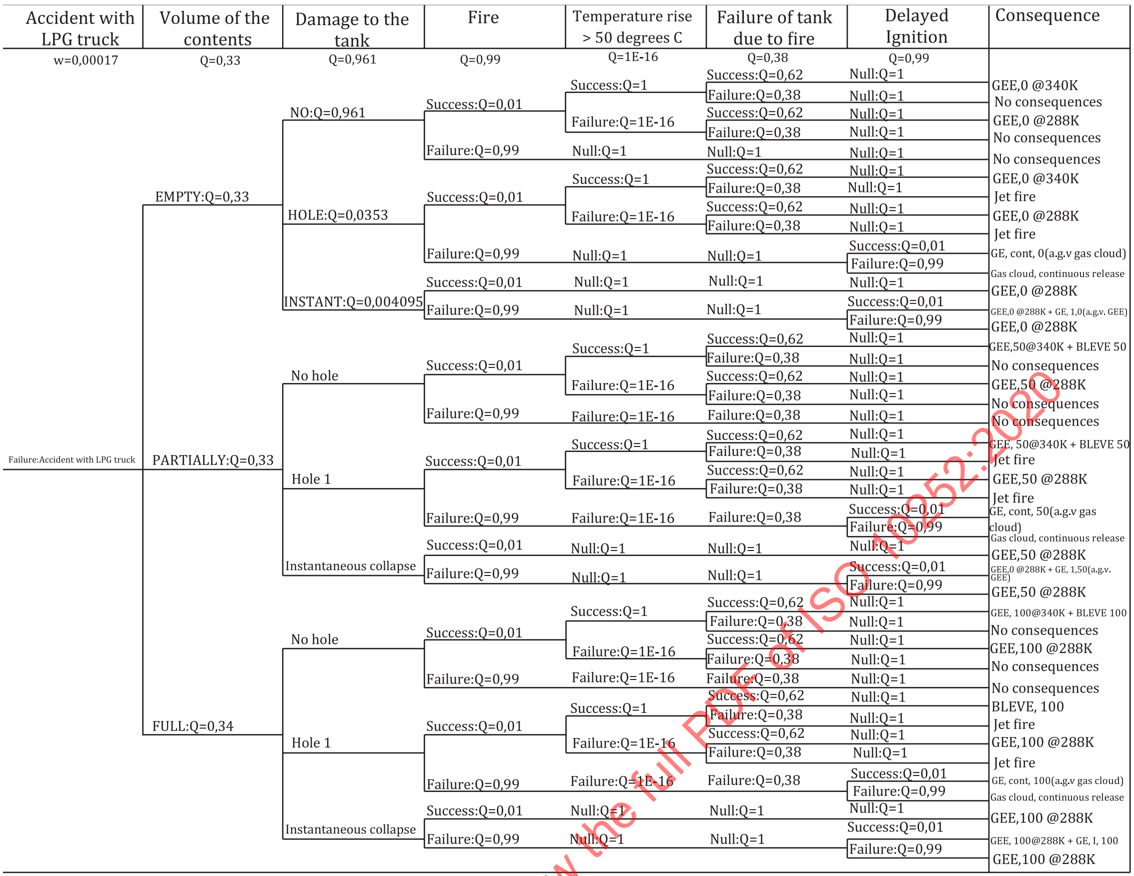
In Figure B.6, an event tree example is presented for explosions in a road tunnel, resulting from an accident with an LPG truck (see References [79] to [82]). The study is based on an "average" Dutch tunnel, with a length of 1 000 m and the location of the explosion is in the centre. Depending on the circumstances, the following types of gas explosions can occur: a standard gas explosion, a gas expansion explosion and a boiling liquid expanding vapor explosion. The notation used in the figure and in Table B.1 is:

— GE I 50, a gas explosion, tank filled for 50 %, instantaneous ignition,

— GEE 100@288 K, a gas expansion explosion, tank filled for 100 %, temperature 288 K,

— BLEVE 100, a BLEVE (boiled liquid expanding vapor explosion) for a tank filled 100 %.

In References [79] to [82], estimates based on calculations and expert judgement are given for the peak pressure and the impulse of the resulting scenarios. Combining the resulting scenario loads and the corresponding discrete probabilities, a continuous probability distribution for the load may be constructed for design purposes. The use of a Gumbel distribution is recommended.



NOTE Numbers are given for illustrative purposes only.

Figure B.6 — Event tree example for explosion in a tunnel

Table B.1 — Estimates of (max) peak pressure and impulse

Scenario	Consequences	Peak pressure kPa	Specific impulse kPa.s
CQ0	No consequences	0	0
CQ1	Jet fire	0	0
CQ2	Gas cloud, continuous release	0	0
CQ3	Gas cloud, instantaneous release	0	0
CQ4	GEE,0 @288K	200	2
CQ5	GEE,0 @326K	500	6
CQ6	GEE, 50 @288K	300	10
CQ7	GEE, 100 @288K	300	20
CQ8	GEE, 0 @288K + *GE, I,0 (a.g.v. GEE)*	575	38
CQ9	GEE, 50 @326 K + *BLEVE 50*	600	30
CQ10	GEE, 100 @326 K + *BLEVE 100*	600	50
CQ11	GEE, 50 @288K + GE, I, 50 (a.g.v. GEE)	300	20
CQ12	GE, I, 0	575	38
CQ13	GE, I, 50	150	20
CQ14	GE, I, 100	130	17,5
NOTE Numbers are given for illustration purposes only.			

Table B.1 *(continued)*

Scenario	Consequences	Peak pressure kPa	Specific impulse kPa.s
CQ15	GE, cont, 0 (a.g.v. gas cloud)	0	0
CQ16	GE, cont, 50 (a.g.v.gas cloud)	1 800	275
CQ17	GE, cont, 100 (a.g.v. gas cloud)	1 800	275
CQ18	BLEVE, 0	500	6
CQ19	BLEVE, 50	600	30
CQ20	BLEVE, 100	600	50
CQ21	GEE, 50 @326K	600	30
CQ22	GEE, 100 @326K	600	50
CQ23	GEE, 100 @288 K + GE, I, 100	300	20
NOTE Numbers are given for illustration purposes only.			

B.3 Dust explosions

B.3.1 General

Flammable dust explosions shall be taken into account in the design of buildings and other civil engineering works where relevant. Pipes and ducts for transport and distribution of dust for the supply of industry, which normally are accessible for maintenance, should normally be designed to resist the anticipated overpressure of a possible dust explosion.

NOTE 1 Principal ignition sources for dust explosion are: flames and direct heat; hot work; incandescent material; hot surfaces; electrostatic sparks; friction sparks; impact sparks; self-heating; static electricity; and lightning.

In considering dust explosion pressure, the context of the accidental action, e.g. whether within a silo/container, a duct or external to the structure, shall be taken into account. The design pressure for a structure during an internal explosion can be reduced by the use of vents that give way at a lower overpressure than the structure itself, and for this purpose vents shall be designed as given in this document.

The influence on the magnitude of an explosion of cascade effects from several connected rooms filled with explosive dust is also not covered in this document.

NOTE 2 The pressure generated by an explosion depends primarily on the type and the percentage of dust in the air, its uniformity of spread, initial turbulences, initial temperature, initial pressure, the location of the ignition source, the presence of obstacles in the enclosure, the size, the shape and the strength of the enclosure in which the explosion occurs, and the amount of venting or pressure release that can be available. The presence of flammable gasses, inert gas or inert dust too can influence the maximum pressure and maximum rate of pressure rise. The maximum explosion pressure is essentially independent of the vessel size, provided heat effects can be disregarded.

For construction works classified as CC1, no specific consideration of the effects of an explosion should be necessary other than to comply with the rules for connections and interaction between components provided in the structural design standards for various materials.

For construction works classified as CC2 or CC3, key elements of the structure should be designed to resist actions by either using an analysis based upon equivalent static load models, or by applying prescriptive design/detailing rules. Additionally, for structures classified as CC3, a dynamic analysis should be used.

NOTE 3 Advanced design for dust explosions generally include one or more of the following aspects[83] to [92]:

— explosion pressure calculations, including the effects of confinements and venting panels;

— dynamic non-linear structural calculations;

— probabilistic aspects and analysis of consequences;

— economic optimisation of mitigating measures.

The explosion pressures provided in this document can be considered as nominal values.

The explosive pressure should be assumed to act effectively simultaneously on all of the bounding surfaces of the enclosure in which the explosion occurs.

B.3.2 Dust explosions in rectangular compartments

B.3.2.1 Rectangular volumes (regular)

Given an amount of venting area available, the design value p_d for the maximum pressure, developed in ~~a~~ vented rectangular compartment for dust explosions may be determined by solving for p_d using empirical Formula (B.3). This Formula applies to individual compartments, not connected to others, with vents that do not have discharge ducts.

$$A_v = \left[4{,}485 \times 10^{-8}\, p_{max} K_{St} p_d^{-0{,}569} + 0{,}027 \left(p_{stat} - 10\right) p_d^{-0{,}5}\right] \frac{V^{0{,}753}}{E_{eff}} \tag{B.3}$$

where

A_v is the venting area, in m^2;

V is the volume of a room, vessel, bunker, in m^3;

K_{St} is the deflagration index of a dust cloud, in kN/m·s;

p_{max} is the maximum pressure of an explosion of the dust, in kN/m^2;

p_{stat} is the static activation pressure of the vent area, in kN/m^2;

p_d is the design value of the pressure in the vented vessel, in kN/m^2;

E_{eff} is the vent efficiency ratio (see EN 14979).

NOTE 1 K_{St} is also called dust explosibility constant (see EN 14034-2 and EN 14491). It is a measure of the maximum rate of pressure rise during an explosion test and normalised to the vessel volume. Some examples are given in Table B.2. For more information, see Reference [96].

Formula (B.3) is not valid if any parameter falls outside of the ranges given below:

— $0{,}1\ m^3 \le V \le 10\ 000\ m^3$,

— $L_3/D_E \le 2$, where L_3 is the largest dimension of the compartment and D_E is the equivalent diameter based on the equal area criterion (not the hydraulic diameter). For a rectangular compartment, $D_E = 2(L_1 \times L_2/\pi)^{0,5}$, where L_1 and L_2 are the other two dimensions of the room,

— $10\ kN/m^2 \le p_{stat} \le 100\ kN/m^2$, rupture disks and panels with low mass which respond almost without inertia,

— $10\ kN/m^2 \le p_d \le 200\ kN/m^2$,

— $500\ kN/m^2 \le p_{max} \le 1\ 000\ kN/m^2$ for $1\ 000$ kN/m·s $\le K_{St} \le 30\ 000$ kN/m·s,

— $500\ kN/m^2 \le p_{max} \le 1\ 200\ kN/m^2$ for $30\ 000$ kN/m·s $\le K_{St} \le 80\ 000$ kN/m·s.

Values of p_{max} and K_{St} may be experimentally determined by standard methods (See ISO 6184-1) for each type of dust.

NOTE 2 Dust explosibility (K_{St}) data can show significant variability, e.g. ±15 %.

NOTE 3 The value of K_{St} depends on factors such as the chemical composition, particle size and moisture content.

NOTE 4 p_{max} can depend on the test vessel used (see ISO 6184-1).

NOTE 5 In dust explosions, pressures reach their maximum value within a time span in the order of 20 ms to 50 ms. The subsequent decline to ambient pressure strongly depends on the venting device and the geometry of the enclosure.

B.3.2.2 Design values for pressures in elongated rectangular rooms

If Formula (B.3) is to be used for a much elongated rectangular compartment where $L_3/D_E \geq 2$ (i.e. in relation to the restriction in B.3.2.1), the vent area shall be increased by ΔA_v (m^2) given by Formula (B.4).

$$\Delta A_v = A_v\left(-4{,}305\log p_d + 9{,}368\right)\log\left(L_3 / D_E\right) \tag{B.4}$$

The equivalent diameter D_E is an equal area criterion (not the hydraulic diameter). The equivalent diameter D_E of a rectangular duct, for the same area, is given by:

$$D_E = 2\left(L_1 \times L_2 / \pi\right)^{0{,}5} \tag{B.5}$$

where L_1 and L_2 are the two smaller dimensions and define the cross-section of the duct.

NOTE 1 Where $L_3/D_E \geq 5$, the structure is to be considered as a duct. See B.3.3.

NOTE 2 The maximum explosion pressure in elongated compartments tends to be lower than in compartments with relatively small aspect ratios, with a higher heat loss.

Table B.2 — p_{max} and K_{St} values for explosive-dust

Type of dust	p_{max} kN/m^2	K_{St} kN/m·s
Brown coal	810 to 1 000	18 000
Cellulose	800 to 980	27 000
Coffee		9 000
Corn, corn crush		12 000
Corn starch		21 000
Grain		13 000
Milk powder	810 to 970	16 000
Mineral coal		13 000
Mixed provender		4 000
Paper		6 000
Pea flour		14 000
Pigment	650 to 1 070	29 000
Rubber	740	14 000
Rye flour, wheat flour		10 000
Soya meal		12 000
Sugar	820 to 940	15 000
Washing powder		27 000

Table B.2 *(continued)*

Type of dust	p_{max} kN/m^2	K_{St} kN/m·s
Wood, wood flour	770 to 1 050	22 000

B.3.3 Dust explosions in ducts

B.3.3.1 General

Where the length-to- (equivalent) diameter ratio of the structure L/D is larger than 5, it shall be considered and designed as a duct.

NOTE 1 For L/D ratios between 3 and 6, the methods given for elongated vessels can be used but they are increasingly conservative.

When determining the design pressure and vent areas, the dimensions of the duct and its layout (i.e. closed at both sides, open at both sides, open at one side, closed at the other) and whether obstacles are built within the duct influence the combustion and the resulting pressures shall be taken into account.

NOTE 2 A higher pressure is experienced when the pressure wave of a deflagration is obstructed by walls or other fixed structures (for example the end wall of a duct). The pressure wave is reflected at such a location and the pressure at the flange of a duct end can be up to three times as high as the pressure at the side wall of a straight duct.

When designing for venting, the vent area should be at least equal to the cross-sectional area of the duct at each venting location, provided at distances less than the critical distance L (see B.3.3.2). Multiple venting locations can be provided within the critical distance L and the vent area of several openings should be determined by adding the individual areas.

Vents should be placed close to possible leakage or ignition sources. If only a single vent is used, it should be placed close to the ignition source. However, if the ignition source is not known, the vent should be placed near the mid-length of the duct.

Vents should not be placed in regions where people can be endangered.

The mass per unit area of a deflagration vent closure should not exceed 12,2 kg/m^2.

The methods given below shall be used only with ducts operating at or near the atmospheric pressure.

The methods given below are applicable for operational pressures less than or equal to 20 kN/m^2.

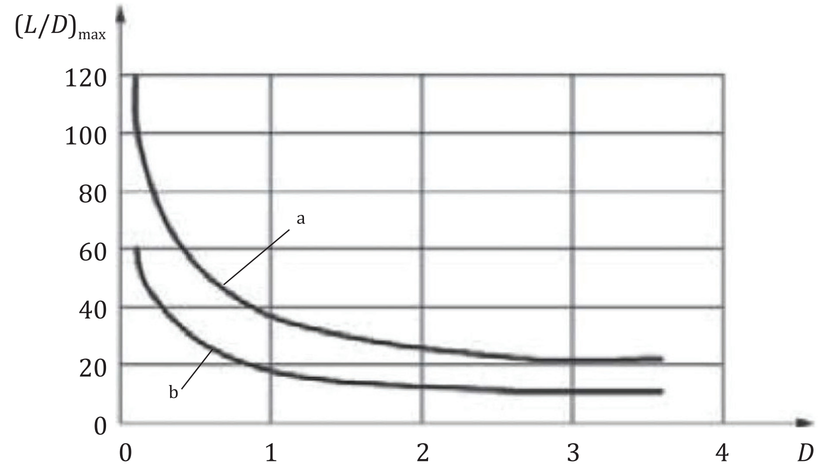
NOTE 3 The applicability of the methods given here is restricted by the K_{St} value.

The methods given in this document shall be applied only to smooth straight ducts and vessels closed at one end and open at the other or those vented at least at intervals smaller than the critical distance.

B.3.3.2 Critical distance

The critical distance L to prevent detonation in a smooth straight pipe or duct where one end is open may be determined from Figure B.7, which is based on the hydraulic mean diameter of the vent.

NOTE By providing sufficient vent areas at a distance less than the critical distance (length) L, deflagration can be prevented from transitioning into a detonation.



Key

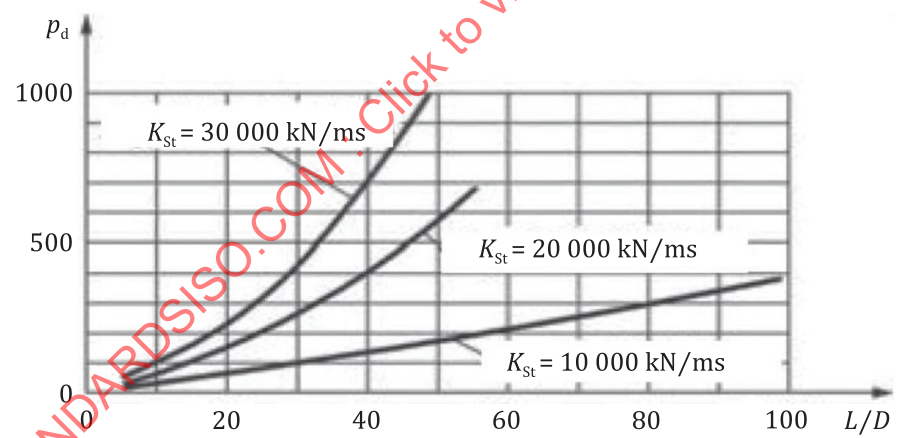
upper curve dust with $K_{st} \leq 20\ 000$ kN/(m·s)

lower curve dust with $K_{st} > 20\ 000$ kN/(m·s)

NOTE L is the distance between deflagration vents or length of duct with one end open. The model is valid only if the initial flow velocity is less than or equal to 2 m/s.

Figure B.7 — Maximum allowable length of smooth, straight pipe or duct closed at one end and vented at the other, to prevent detonation

The design overpressure p_d in a vented pipe containing dust may be determined from Figure B.8 when the initial flow velocity is less than 2 m/s.



NOTE Valid only if the initial flow velocity is less than 2 m/s.

Figure B.8 — Design overpressure P_d (kN/m^2) for vented pipes containing dust

To reduce the deflagration design overpressure to below 20 kN/m^2, the spacing of multiple vents for a dust with a K_{St} value of less than 30 000 kN/m·s and an initial flow velocity of between 2 m/s and 20 m/s can be selected from Figure B.9.

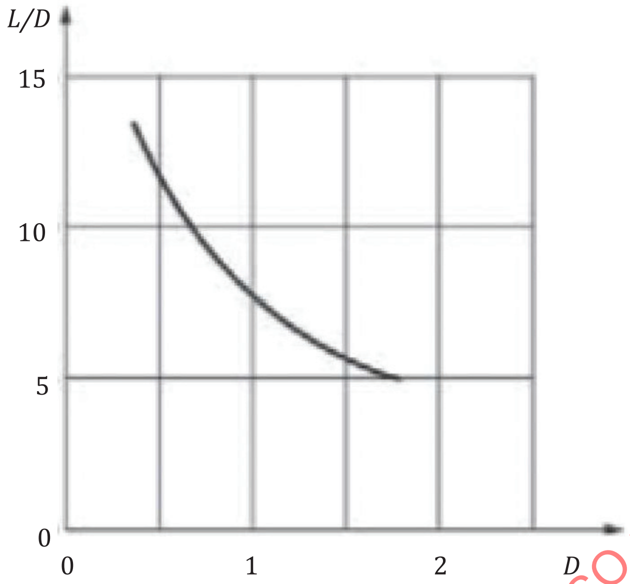


Figure B.9 — L/D values for dust with a K_{St} value of less than 30 000 kN/ms and an initial flow velocity of between 2 m/s and 20 m/s, for design overpressure <20 kN/m^2

B.3.4 External dust explosions

In the absence of design formulae, computer codes may be used in the assessment of the potential for external explosion and its design overpressure.

B.4 External vapour cloud and high energy explosions

B.4.1 Sources and scenarios of external explosions

External vapour cloud explosions can result from:

— warehouses/no explosives,

— stationary pressurised tanks and vessels,

— stationary atmospheric tanks and vessels,

— gas cylinders,

— pipes,

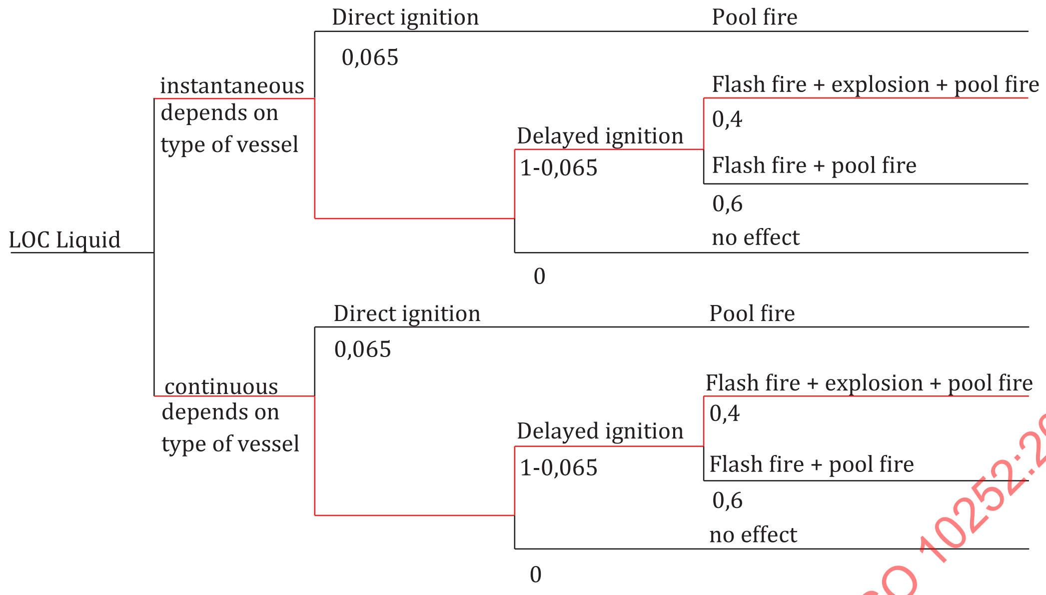
— road tankers,

— tank wagons (trains),

— ships.

In the second group, the length dimension of the pipe or (water/rail) way has to be taken into account.

The initiating event for a vapour cloud explosion in general is a loss of containment (LOC) of the explosive material. The LOC can have several causes in itself. Depending on the dimensions of the hole, the out flow can be characterized as calm/continuous or instantaneous. If there is an ignition, immediate ignition increases the probability of a fire and a delayed ignition increases the probability of an explosion. For each of the above situations, event trees describing the various events can be found in the literature[97][98][99]. An example is presented in Figure B.10.



NOTE Numbers are based on Reference [99]. The BLEVE has been neglected.

Figure B.10 — Event tree: loss of containment (LOC) from an atmospheric flammable fluid tank containing ethanol

High-energy explosions can result from storage, handling, or transport of explosives in the vicinity of a structure under consideration.

B.4.2 Models for loads on structures

Given the event of an explosion, the free air pressure p depends on the amount of available energy E and the distance R and can be found using models like the Multi Energy, CFD or TNT equivalent B2, B8. These models are not discussed here in detail. Computer models exist to find the load distribution on all facades of a building as a function of time.

Obstacles in the vicinity of the blast source are known to have a significant effect on the blast wave in some cases; in particular in case of delayed ignition where the obstacle is situated inside the vapour cloud at the moment of ignition. This effect, although significant, is still difficult to quantify with a simple analytical model. For this reason, and for the time being, this effect is not explicitly modelled but considered to be part of the random scatter.

Obstacles in between the building to be designed and the blast source — i.e. other than obstacles inside the vapour cloud — can have a "shielding effect" on the building. However, research has shown that the shielding effect is only significant if the obstacle is situated very close to the building and is substantially large. For these reasons, the presence of shielding obstacles is not considered.

In the free air, a pressure wave from an explosion theoretically is shaped like in Figure B.11 with a positive and negative phase. For most applications, it is sufficient to translate the free pressure diagram to a triangle having a peak load value p and a duration, Δt, as indicated in Figure B.11.

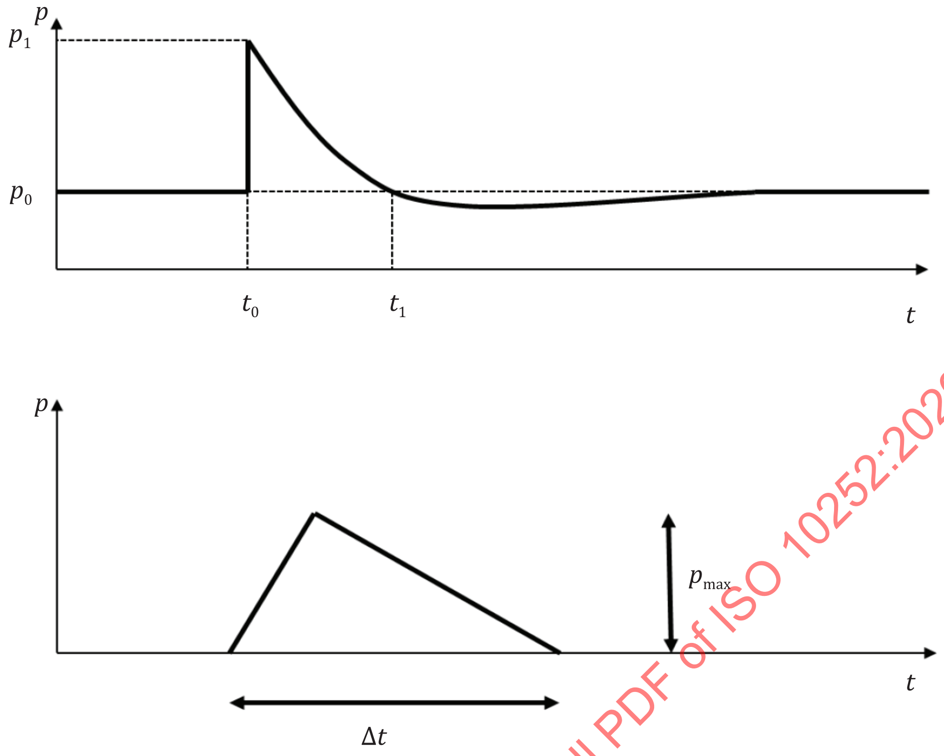


Figure B.11 — Realistic pressure wave and triangular simplification (relative to the atmospheric pressure)

EXAMPLE For an explosion having an effective energy E = 1,3 GJ and a distance R = 100 m, a simple TNT model according to Reference [70] or [97] results in a free air peak pressure p_{max} = 8 kPa and a duration Δt = 35 ms.

The rise time for vapour explosions often is comparable to the decay time. For high-energy explosions, the rise time is often taken as instantaneous. However, the actual shape of the impulse constructed from the pressure history often is less important than the total magnitude of the impulse, particularly when the duration of the load is short compared to the periods of the excited vibration modes of the structure.

For some applications, it can be convenient to work with pressure and impulse; the impulse value in case of a triangle is equal to;

$$i = 0{,}5 p_{max}\,\Delta t \tag{B.6}$$

When the blast wave reaches the building, a complex pattern of reflection and interaction occurs. In some cases, this can lead to much higher pressures on the building facades than the free value. For larger distances, the factor is minimal 2, but maybe more if the structure is close to the centre of the explosion.

Blast effects from high-energy exterior explosions shall be calculated approximately with the following empirical procedure when:

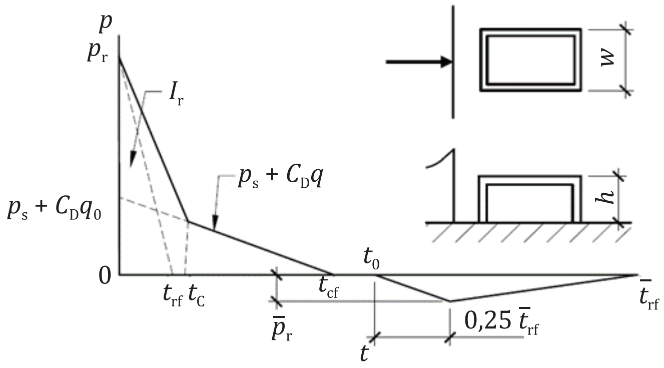
a) The blast occurs on or near the ground surface external to the structure.

b) The blast loading is not significantly affected by surrounding structures or terrain.

c) The structure has no unusual irregularities in spatial form.

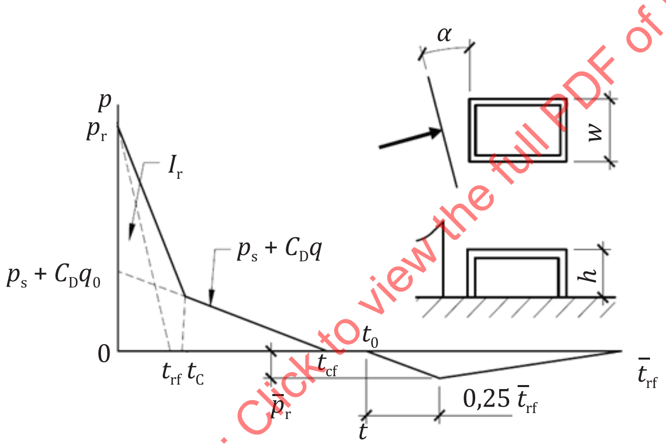
d) The scaled distance from the blast to the structure is within the scaled ranges of the figures contained herein.

The loading on directly loaded surfaces shall be idealized as shown in Figures B.12 and B.13 for normal reflection and oblique reflection.



NOTE Arrow is direction of shock wave.

Figure B.12 — Front wall loading, normal reflection



NOTE Arrow is direction of shock wave.

Figure B.13 — Front wall loading, oblique reflection

The equivalent mass of trinitrotoluene (TNT), W_e, shall be determined by multiplying the effectiveness factor from Table B.3 by the mass of the actual explosive material.

Table B.3 — Equivalent TNT masses for air blast in free air

Explosive	Density kg/m^3	Equivalent mass for pressure	Equivalent mass for impulse	Overpressure range kPa
Amatol (50/50)	1 584	0,97	0,87	NA[a]
Ammonia dynamite (50 % strength)	NA[a]	0,90	0,90[b]	NA[a]
Ammonia dynamite (20 % strength)	NA[a]	0,70	0,70[b]	NA[a]
ANFO (94/6 ammonium nitrate/fuel oil)	NA[a]	0,87	0,87[b]	27,56 to 6 890
AFX-644	1 744	0,73[b]	0,73[b]	NA[a]

[a] NA, data not available.

[b] Value is estimated.

Table B.3 *(continued)*

Explosive	Density kg/m³	Equivalent mass for pressure	Equivalent mass for impulse	Overpressure range kPa
AFX-920	1 584	1,01[b]	1,01[b]	NA[a]
AFX-931	1 616	1,04[b]	1,04[b]	NA[a]
Composition A-3	1 648	1,09	1,07	27,56 to 351,39
Composition B	1 648	1,11	0,98	27,56 to 351,39
		1,20	1,30	689 to 6 890
Composition C-3	1 600	1,05	1,09	NA[a]
Composition C-4	1 584	1,20	1,19	68,9 to 1 378
		1,37	1,19	
Cyclotol (75/25 RDX/TNT)	1 712	1,11	1,26	NA[a]
(70/30)	1 728	1,14	1,09	27,56 to 351,39
(60/40)	1 744	1,04	1,16	NA[a]
DATB	1 792	0,87	0,96	NA[a]
Explosive D	1 712	0,85[b]	0,81	6,89 to 303,16
Gelatin dynamite (50% strength)	NA[a]	0,80	0,80[b]	NA[a]
Gelatin dynamite (20% strength)	NA[a]	0,70	0,70[b]	NA[a]
H-6	1 760	1,38	1,15	27,56 to 702,78
HBX-1	1 760	1,17	1,16	27,56 to 137,8
HBX-3	1 840	1,14	0,97	27,56 to 172,25
HMX	NA[a]	1,25	1,25[b]	NA[a]
LX-14	NA[a]	1,80	1,80[b]	NA[a]
MINOL II	1 824	1,20	1,11	20,67 to 137,8
Nitrocellulose	1 648 to 1 696	0,50	0,50[b]	NA[a]
Nitroglycerin dynamite (50 % strength)	NA[a]	0,90	0,90[b]	NA[a]
Nitroguanidine (NO)	1 712	1,00	1,00[b]	NA[a]
Nitromethane	NA[a]	1,00	1,00[b]	NA[a]
Octol (75/25 HMX/TNT)	1 808	1,02	1,06	NA[a]
(70/30)	1 408	1,09	1,09[b]	6,89 to 303,16
PBX-9010	1 792	1,29	1,29[b]	27,56 to 213,59
PBX-9404	1 808	1,13	1,13[b]	27,56 to 689
		1,70	1,70	689 to 6 890
PBX-9502	1 888	1,00	1,00	NA[a]
PBXC-129	1 712	1,10	1,10[b]	NA[a]
PBXN-4	1 712	0,83	0,85	NA[a]
PBXN-107	1 632	1,05[b]	1,05[b]	NA[a]
PBXN-109	1 664	1,05[b]	1,05[b]	NA[a]
PBXW-9	NA[a]	1,30	1,30[b]	NA[a]
PBXW-125	1 792	1,02[b]	1,02[b]	NA[a]

[a] NA, data not available.

[b] Value is estimated.

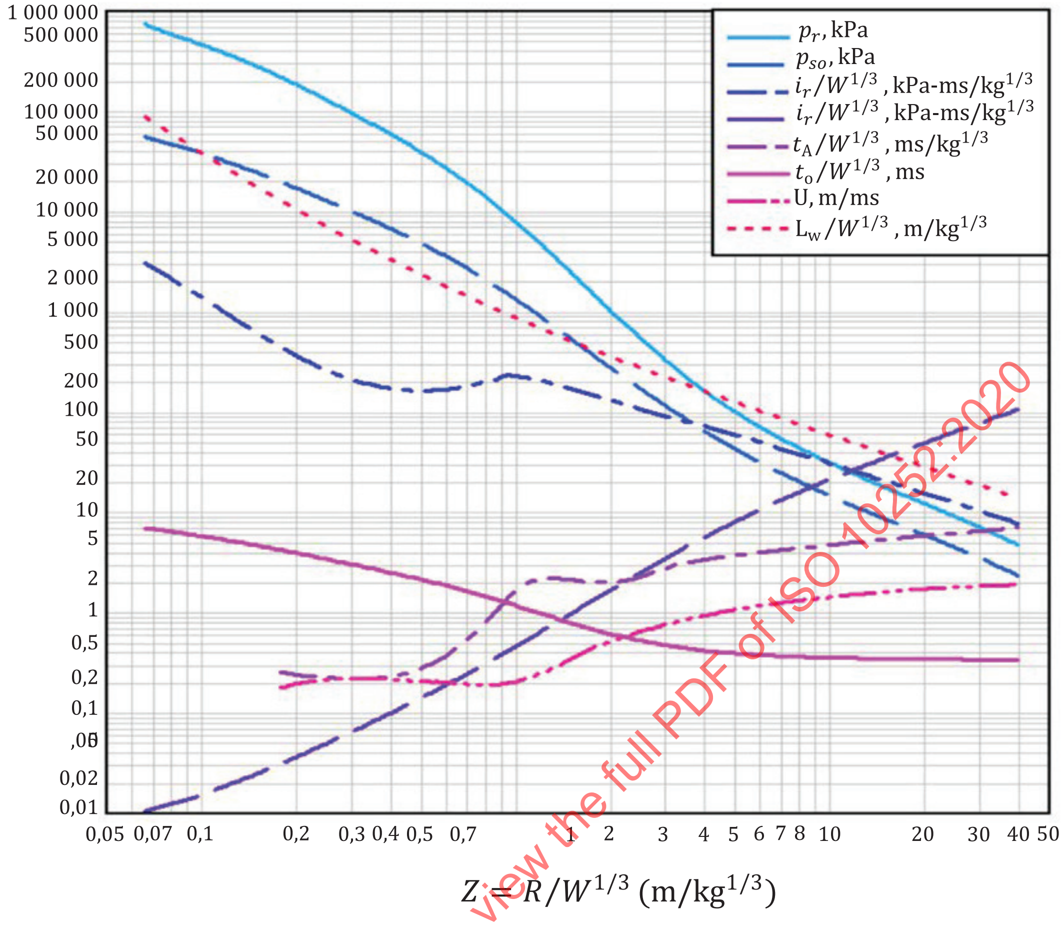
Table B.3 *(continued)*

Explosive	Density kg/m³	Equivalent mass for pressure	Equivalent mass for impulse	Overpressure range kPa
Pentolite (cast)	1 632	1,42	1,00	27,56 to 689
	1 680	1,38	1,14	27,56 to 4 134
	NA[a]	1,50	1,00	689 to 6 890
PENT	1 760	1,27	1,27[b]	27,56 to 689
Picrotol (52/48 Ex D/TNT)	1 632	0,90	0,93	27,56 to 4 099,55
RDX	NA[a]	1,10	1,10[b]	NA[a]
RDX/Wax (98/2)	1 920	1,16	1,16[b]	NA[a]
RDX/AL/Wax (74/21/5)	NA[a]	1,30	1,30[b]	NA[a]
TATB	NA[a]	1,00	1,00[b]	NA[a]
Tetryl	1 728	1,07	1,07[b]	20,67 to 137,8
Tetrytol (75/25 Tetryl/TNT)	1 584	1,06	1,06[b]	NA[a]
TNETB	1 696	1,13.	0,96	27,56 to 689
TNETB/Al (90/10)	1 744	1,23	1,11	27,56 to 689
(78/22)	1 184	1,32	1,32[b]	NA[a]
(65/35)	1 232	1,38	1,38[b]	NA[a]
TNT	1 632	1,00	1,00	Standard
Tropex	1 840	1,23	1,28	6,89 to 303,16
Tritonal (80/20 TNT/AL)	107	1,07	0,96	27,56 to 689

[a] NA, data not available.

[b] Value is estimated.

The incident and normally reflected shock wave parameters for the positive phase of the loading shall be determined from Figure B.14.



Key

R minimum distance from detonation to the component being examined

W equivalent TNT mass of the explosive material in (kg)

p_{so} peak side-on or incident overpressure

i_s incident impulse, per unit area

t_o duration

p_r peak normally reflected overpressure

i_r normally reflected impulse, per unit area

Us shock front velocity

Figure B.14 — Positive shock phase parameters for explosions of hemispherical TNT charges on the surface (USA Dept of Defense, Structures to Resist the Effects of Accidental Explosions, UFC 3-340-02, 2008, SCE/SEI 59-11)

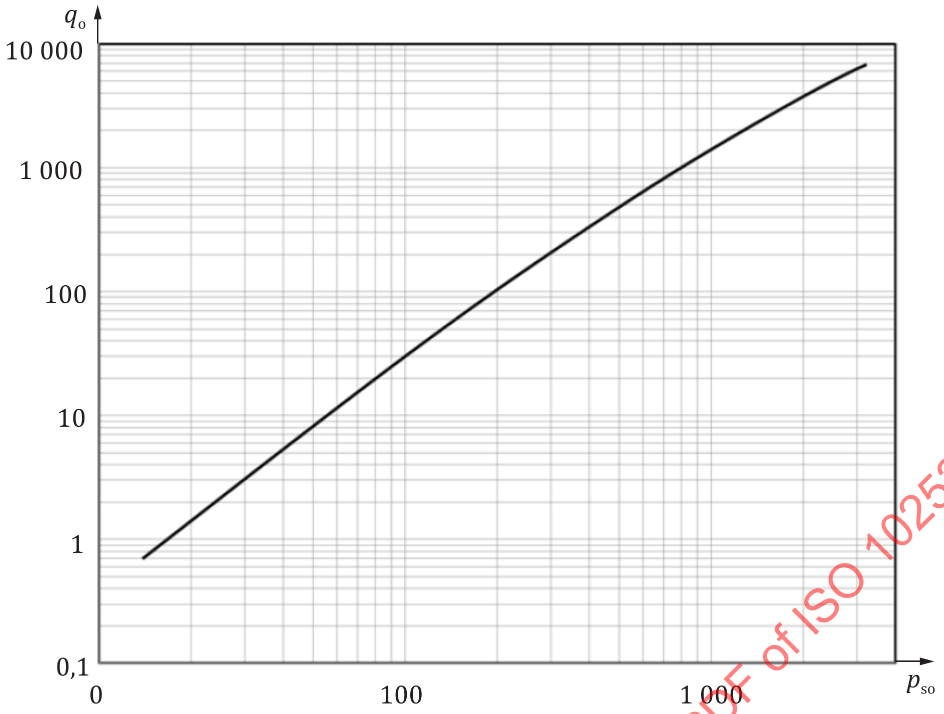
The duration of the equivalent triangular incident loading shall be determined from Formula (B.7):

$$t_{of} = 2i_s / p_{so} \tag{B.7}$$

The duration of the equivalent triangular normally reflected loading shall be determined from Formula (B.8):

$$t_{rf} = 2i_r / p_r \tag{B.8}$$

The peak dynamic pressure, q_o, shall be determined as a function of the peak side-on overpressure from Figure B.15.



Key

p_{so} peak side-on or incident overpressure (kPa)

q_o peak dynamic pressure (kPa)

Figure B.15 — Peak dynamic pressure q_o as a function of p_{so} (ASCE/SEI 59-11)

The average clearing time to relieve the reflected overpressure shall be determined from Formula (B.9):

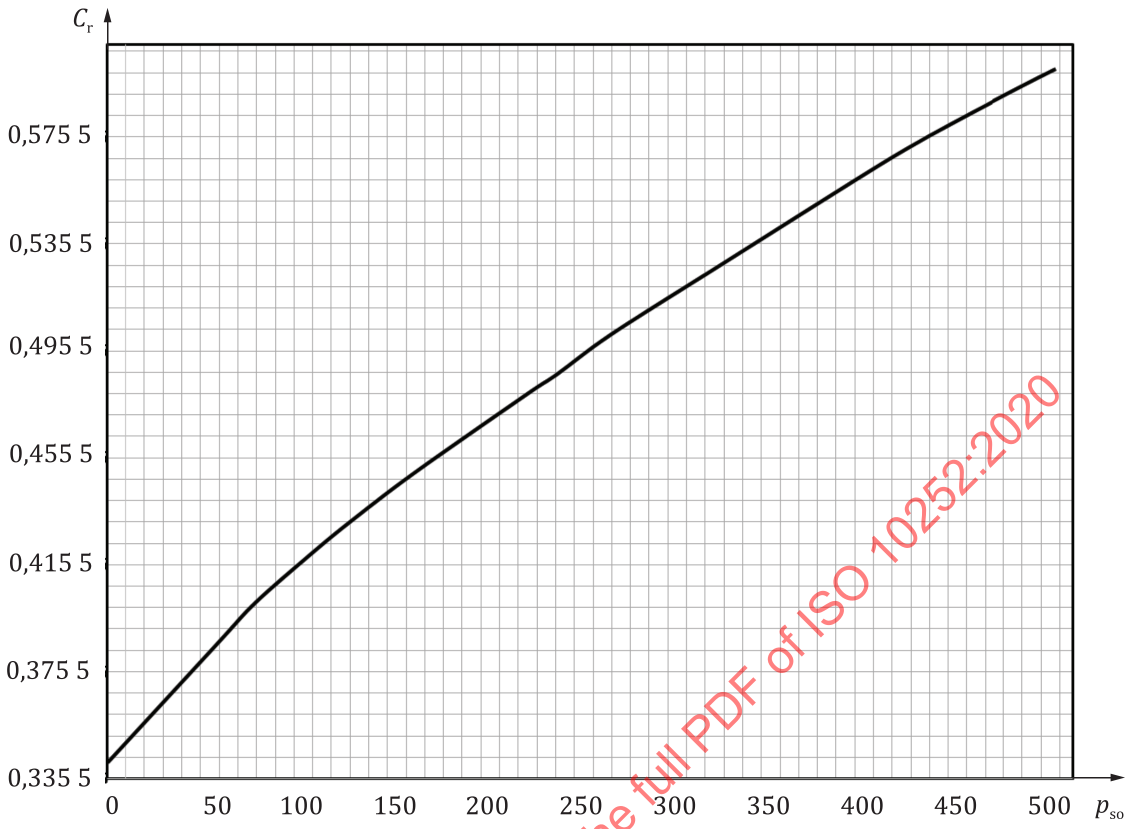
$$t_c = 4hw / (w + 2h) c_r \tag{B.9}$$

where

h is structure height;

w is structure width;

c_r is the sound velocity from Figure B.16.



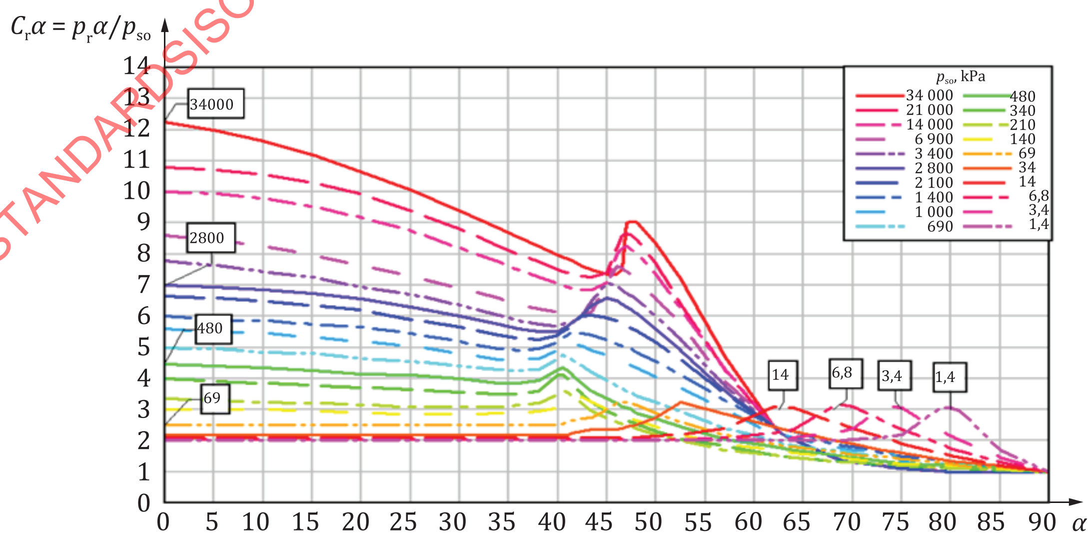
Key

p_{so} peak side-on or incident overpressure (kPa)

c_r sound velocity (m/ms)

Figure B.16 — Sound velocity in reflected overpressure region (ASCE/SEI 59-11)

The direct drag load coefficient should be taken as C_D = 1. The positive phase loading should be the triangular or bilinear function thus obtained that has the lesser impulse. For oblique reflections at the incident angle α, the reflected overpressure coefficient, $C_{r\alpha}$, shall be obtained from Figure B.17.



Key

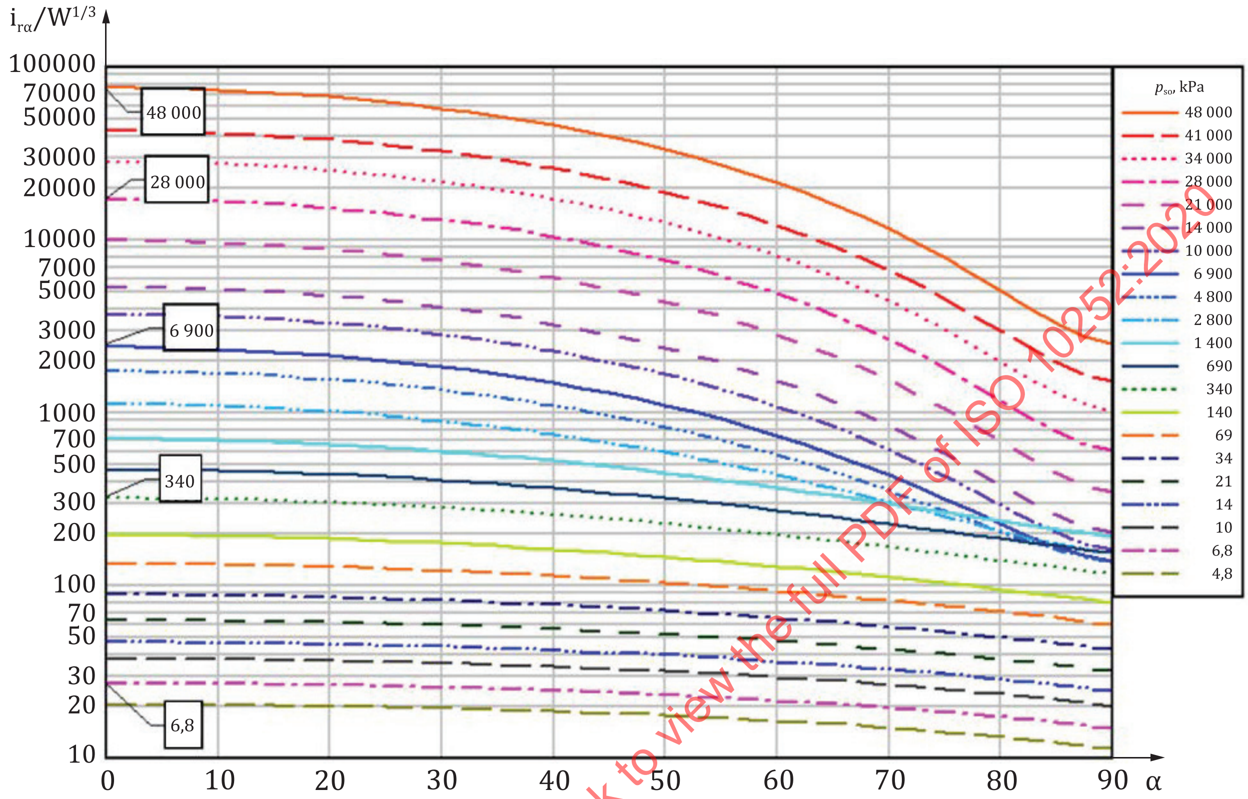
α = angle of incidence

Figure B.17 — Reflected overpressure coefficient $C_{r\alpha} = p_{r\alpha}/p_{so}$ (ASCE/SEI 59-11)

The oblique peak reflected overpressure shall be determined from Formula (B.10):

$$p_{ra} = C_{ra} \, p_{so} \tag{B.10}$$

The oblique reflected impulse, $i_{r\alpha}$, shall be determined from Figure B.18.



Key

α angle of incidence

$i_{r\alpha}$ reflected impulse

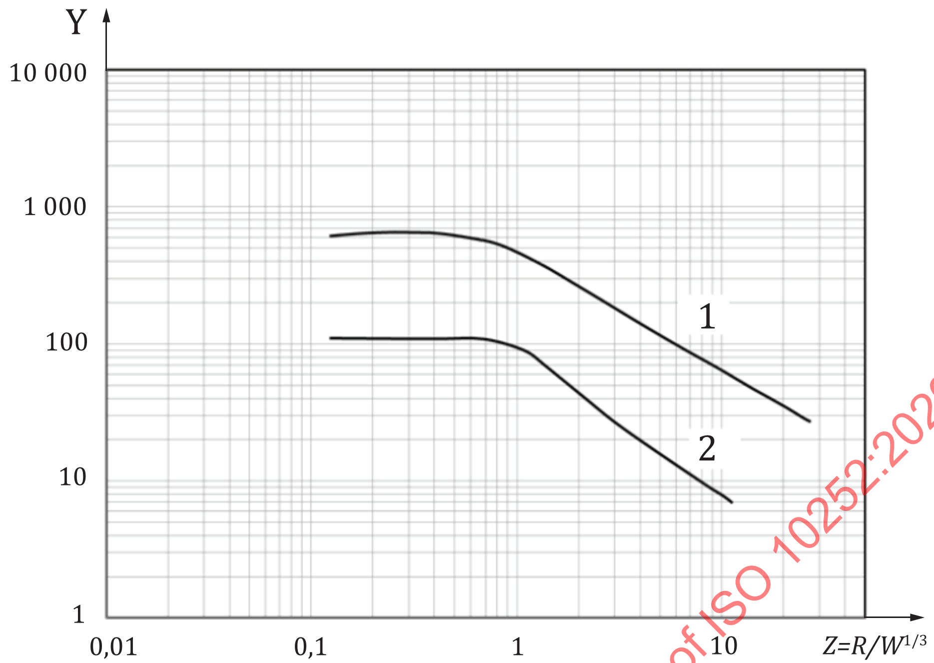
W equivalent TNT mass in (kg)

Figure B.18 — Reflected impulse (ASCE/SEI 59-11)

The duration of the equivalent triangular obliquely reflected loading shall be determined from Formula (B.11):

$$t_{rf} = 2i_{ra} / p_{ra} \tag{B.11}$$

If the negative phase of the loading is to be considered, the parameters shall be determined from Figure B.19.



Key

1 $i_{\bar{R}} / W_e^{1/3}$ (kPa · ms) / kg$^{1/3}$

2 p_r (kPa)

p_r peak normally reflected pressure

i_r normally refelcted impulse per unit area

Z scaled distance (m/kg$^{1/3}$)

Figure B.19 — Negative phase shock parameters for explosions of hemispherical TNT charges on the surface at sea level

The duration of the equivalent triangular negative phase loading shall be determined from Formula (B.12):

$$t_{\bar{r}f} = 2\frac{i_{\bar{r}}}{p_{\bar{r}}} \quad \text{(B.12)}$$

The "rise" time of the negative phase loading shall be $t_{\bar{r}f}/4$.

The loadings on indirectly loaded surfaces shall be calculated using the following procedure. The loading shall be the linear function, $p(t) = p_s + C_D\, q$, shown in preceding charts, in which p_s is the overpressure and q the dynamic pressure. However, for these indirectly loaded roofs, side walls, and rear wall surfaces, the drag coefficient shall be the negative values obtained from Table B.6.

Table B.6 — Side on element dynamic drag coefficients

Peak dynamic pressure q_o	Drag coefficient C_D
0 kPa to 175 kPa	−0,40
175 kPa to 350 kPa	−0,30
350 kPa to 1 000 kPa	−0,20

See Reference [70] for more information.

As an alternative to the approach described above, one may follow rigorous procedures that model the explosive environment and the structural response accurately.

B.4.3 Stochastic model/statistical numbers

B.4.3.1 Occurrence

The likelihood of the loss of containment is defined by the explosion rate λ [1/*a*]; generic data can be found in Reference [97]. This reference also shows data for probabilities of outflow characteristics, immediate or direct ignition and consequences in terms of fire, an explosion of BLEVE (boiling liquid expanding vapour explosion).

Typical resulting values for the explosion are 3×10^{-6}/a for a single installation and 3×10^{-4}/a for a plant.

B.4.3.2 Magnitude

The energy of a stoichiometric mixture is about 3,5 MJ/m^3. Given a large vapour cloud of r = 30 m, we have a gas air volume of V = 50 000 m^3 which correspondents to 200 GJ. In most cases, only a small amount of 2 % to 20 % of this energy is converted into explosion energy, depending on the actual mixture conditions and the blockage properties of the environment. As a result the average effective blast energy E_{eff} is about 10 GJ.

A lognormal or Weibull distribution with V = 1,0 as the coefficient of variation is recommended. The resulting distribution is presented in Figure B.20.

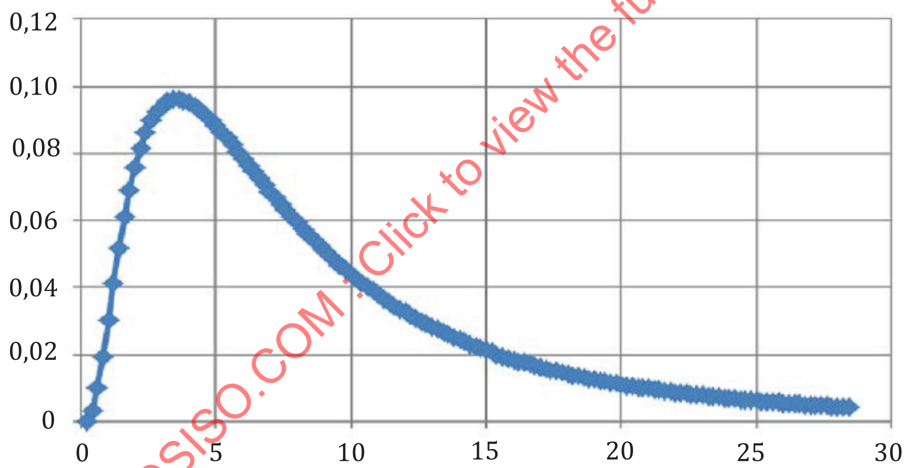


Figure B.20 — Probability density function for the effective amount of energy E_{eff} (GJ)

Assuming a circular shaped industrial area ($A = \pi R_o^2$) and the building of interest in the centre, the probability of R is:

$$f_R(r) = 2\pi r / A = 2r / R_o^2 \quad \text{for } R_{min} < r < R_o \tag{B.13}$$

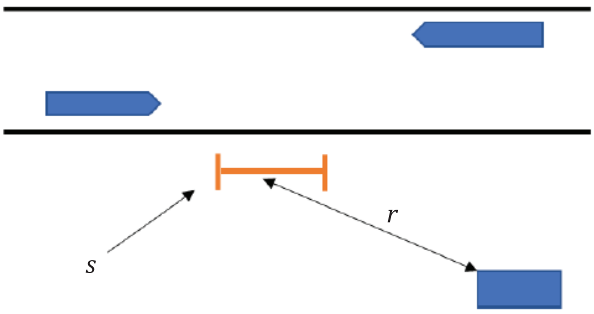
So large values of the distance R are more likely. In addition, a minimum value of e.g. R_{min} = 10 or 20 m can be appropriate.

B.4.3.3 Transport systems

For vapour clouds resulting from transport lines (roads, railways, waterways, pipelines), the cumulative effect of the total line has to be considered.

Transport of explosive materials considered consists of trucks, trains, ships and pipeline transport. In these cases, the occurrence rate λ has to be specified "per transport unit and per km" or "per km pipeline". Given the intensity of the traffic, one can then calculate the expected number of explosions per year.

In the case of transport, the road or river has to be subdivided into a number of segments as indicated in Figure B.21. For each segment, the probability of an explosion may be calculated given the occurrence rate per vehicle kilometer, the number of transport units per year.



Key

s segment of the waterway

r distance from potential accident location to building

Figure B.21 — Procedure in case of transport of explosive material on a waterway

The resulting probability distribution for the pressure at the building site can be found from Formula (B.14):

$$P(p>x)=\sum \lambda_{\mathrm{H}}\,\Delta s\,n\,P(E>E_{xr}) \tag{B.14}$$

where

$P(p > x)$	is the probability of the pressure p exceeding the value x in one year;
λ_{H}	is the probability rate of explosion per vehicle (or ship or train wagon) km;
Δs	is the length of the segment of a road or waterway in (km);
n	is the number of vehicles (ships, train wagons) per year (both directions);
r	is the distance from the heart of a segment to the building;
E	is the amount of energy release given explosion;
E_{xr}	is the amount of energy leading to pressure $p = x$ at distance r.

Typical releases for highway road tankers are in the order of 1 000 kg benzene (18 GJ). For railway tankers, a larger amount may be assumed. A coefficient of variation of $V(E)$ = 1,0 seems reasonable.

Annex C
(informative)

Design for accidental actions

C.1 General design considerations

Strategies for designing structures against identified and unidentified accidental actions should be selected from among those shown in Figure C.1.

According to Clause 5, the design against identified actions (impact, explosions and others where relevant or demanded by local authorities) shall be based on quantitative assumptions and estimates for the loading and a subsequent structural design aiming at limited and local structural damage[COST TU601]. Non-structural risk control measures and full or partial protection systems should be included. The choice of identified accidental actions taken into account, the design values and damage limits depend on the consequence class as described in ISO 2394. For consequence classes 4 and 5, a risk analysis is recommended.

NOTE 1 When designing for robustness, for common structures, the practical way is to select as a start a consequence class based on the expected or maximum consequences of its complete collapse.

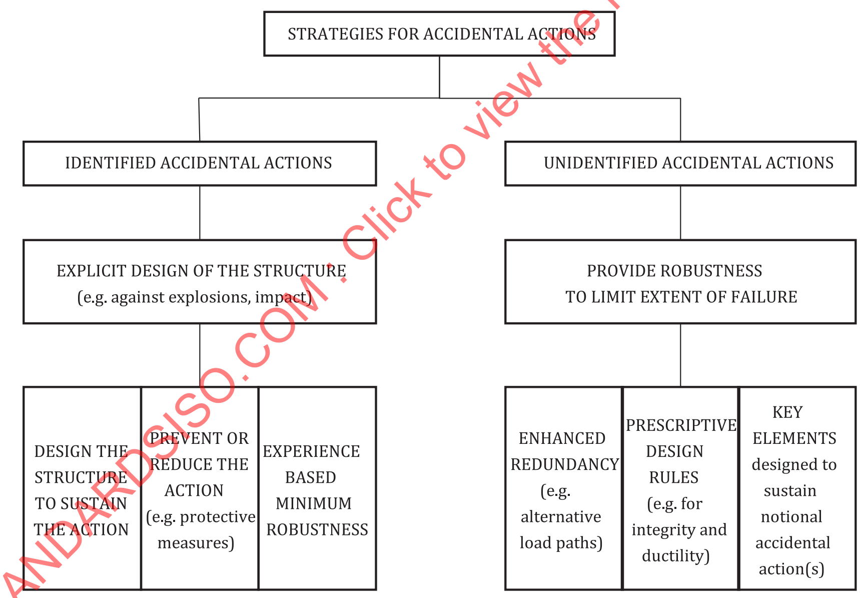


Figure C.1 — Strategies for design against accidental actions[100]

For design against unidentified actions, the following recommendations depending on the consequence class apply:

— Consequence class structures 1 and 2 can be implicitly designed against unidentified actions via the relevant prescriptive design rules to give a level of safety considered acceptable by society. Designs based on prescriptive rules should be carried out in accordance with the relevant International Standard for the particular construction material(s) specified for the building.

— For consequence class 2 buildings, additionally, the provision of effective horizontal ties, or effective anchorage of suspended floors to walls, respectively, is recommended for framed and load-bearing wall construction, as for example available in Eurocode EN 1991-1-7.

