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):

$$E_c = \frac{{m_f}^2}{m_f + m_s} gh \tag{A.23}$$

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:

$$E_c = \frac{{m_f}^2}{m_s} gh \tag{A.24}$$

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.

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

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].

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

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:

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

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);

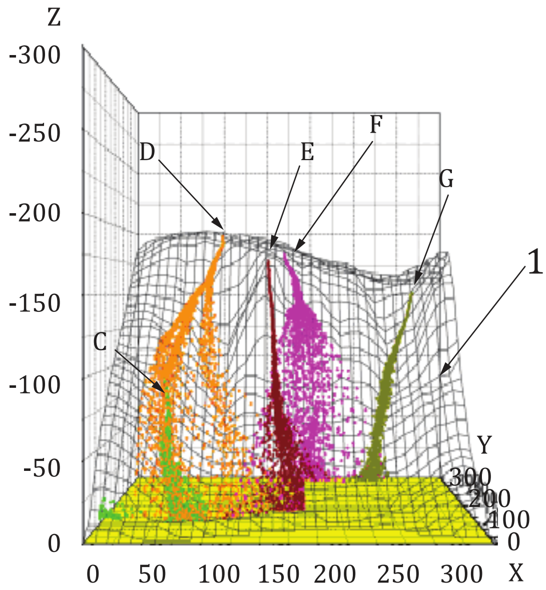
C_c is uniformity coefficient of cushion;

β_r is correction factor concerning cushion
$\beta_r = -5{,}34\ d + 5{,}84$ for $d < 0{,}9$
$\beta_r = 1{,}03$ 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.

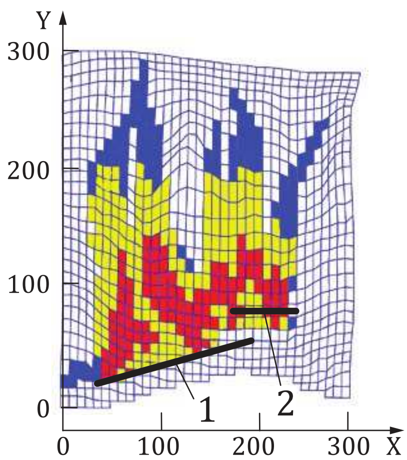
Figure A.27 shows the hazard map of rockfalls based on the kinetic energy of rockfall using trajectory simulation results. Relatively large energies (over 3 000 kJ) are observed at lower slope and even extremely large (more than 6 000 kJ) area is also found. The use of this kind of simulation method 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].

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

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

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

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

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

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;
v_1 and v_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 $p_{red} = p_d$.

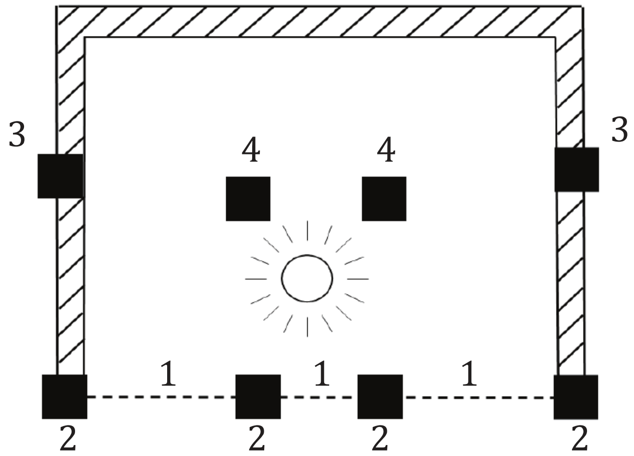
In determining the capacity of a venting panel, account shall be taken of the dimensioning and construction of the supporting frame of the panel.

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.

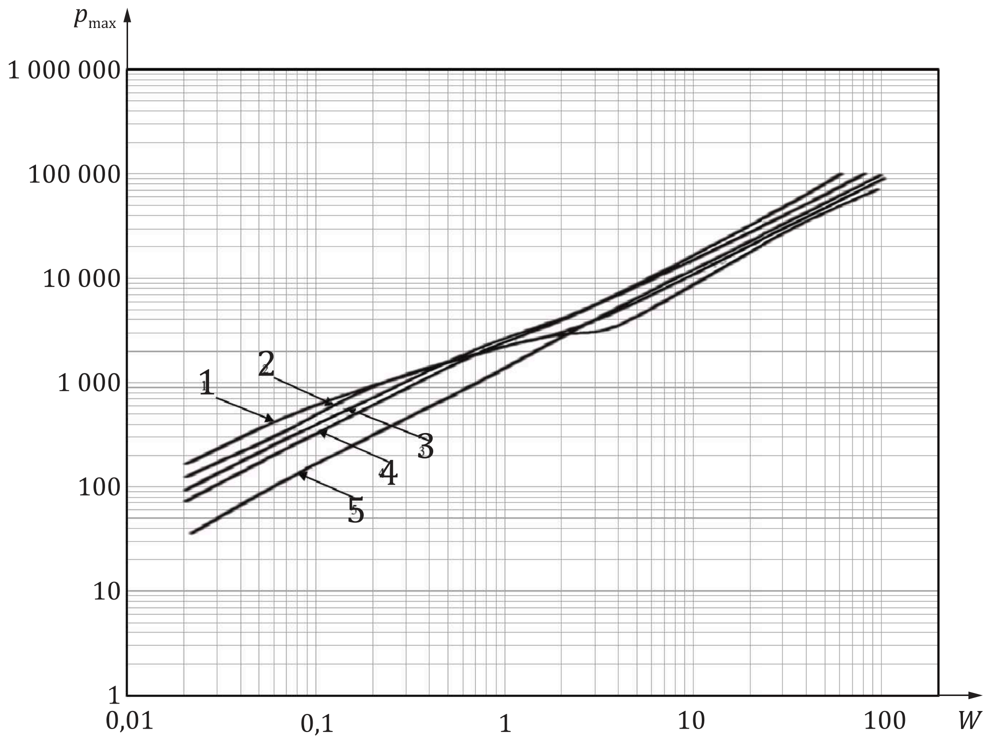
Loads of structural members are not only determined by the peak pressure in the room but also depend on the total configuration. For instance, ignition of a vapour inside a building releases heat that generates overpressure in the confined space. That overpressure drives flow toward vent openings. The 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

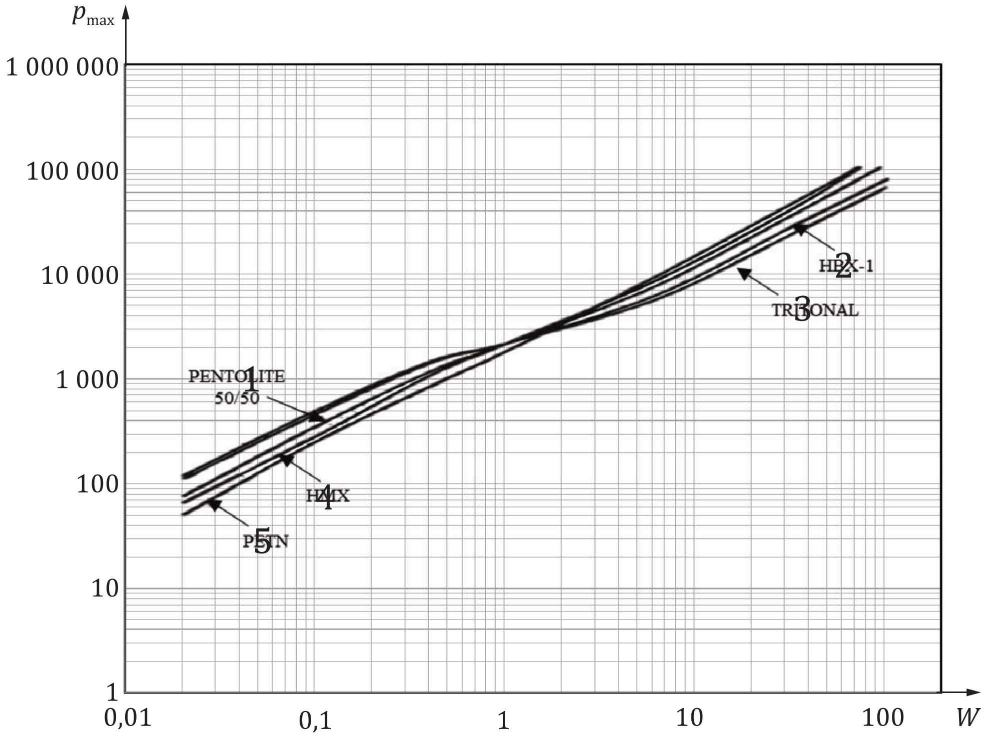
2 = TNT

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.