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Probabilistic study of premature shear failure of slender reinforced concrete one-way slabs subjected to blast loading

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Abstract

Blast-loaded reinforced concrete (RC) slabs should fail under a ductile bending mechanism enabling high energy absorption capacity. Hence, brittle shear failure must be avoided. However, due to the uncertainties related to the materials, geometry, and resistance models, it may be difficult to predict which failure mode will prevail. This study analytically estimated the probability of premature flexural shear failure of slender RC one-way slabs subjected to blast loading considering such uncertainties and using the Monte Carlo method. The resistance models in Eurocode 2 were adopted. Specimens with and without shear reinforcement were analysed. Bending failure was shown to be the most likely failure mode in the studied slabs. However, the probability of shear failure developing before bending failure was still relatively high, particularly for slabs without stirrups. To increase the confidence level concerning the preferred failure mechanism, the article proposes an overstrength factor to magnify the shear demand of the blast-loaded RC slab. Values of the overstrength factor for different target reliability levels were calculated. The study also found that the probability of premature shear failure increased with increasing amount of longitudinal reinforcement and decreasing slenderness. Likewise, greater impulse was found to enhance the risk for shear failure.

Keywords: Reinforced concrete slabs, blast loading, premature shear failure, model uncertainty, Monte Carlo method

1 Introduction

An explosion is a sudden release of energy caused by e.g. detonation of explosive charges or turbulent combustion of fuel-air mixtures. The rapid, major release of energy pushes the surrounding air into motion as a blast wave that propagates outwards from the source of the explosion. When the blast wave impinges on nearby structures, it delivers a high-pressure pulse of short duration, which may cause severe damage or total collapse. For reinforced concrete (RC) structures exposed to such loading, the ability to safely absorb the energy delivered by the blast wave is of utmost importance. The blast wave produces an external work, W_e , which needs to be balanced by an internal work¹, W_i , in the structure: $W_e = W_i$. The total internal work that can be mobilised before collapse depends on both the deformation capacity and the load resistance of the structure [1,2]. However, it is often more practical and economical to design the structure to dissipate the energy imparted by the blast by means of a large inelastic deformation, rather than relying on a large load resistance. Therefore, it is essential to ensure that blast-loaded structures have sufficient plastic deformation capacity to develop the required internal work.

Simply supported, slender RC one-way slabs subjected to blast loading can fail mainly under two different mechanisms: *bending (moment) failure* and *flexural shear failure* (hereinafter also referred to simply as *shear*

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¹ Also commonly referred to as internal strain energy [1]

failure). These two types of failure normally differ substantially in the deformation that the slab can attain before collapsing, as illustrated schematically in Figure 1. A RC slab failing under bending usually exhibits a ductile response with major plastic deformations. Conversely, shear failure is typically characterised by brittle behaviour with limited deformation capacity. Consequently, to achieve high energy absorption capacity in blast-loaded RC slabs, bending failure should arise first, before shear failure occurs. Indeed, the necessity of avoiding premature shear failure of blast-loaded RC elements has been recognised in the literature [3,4].

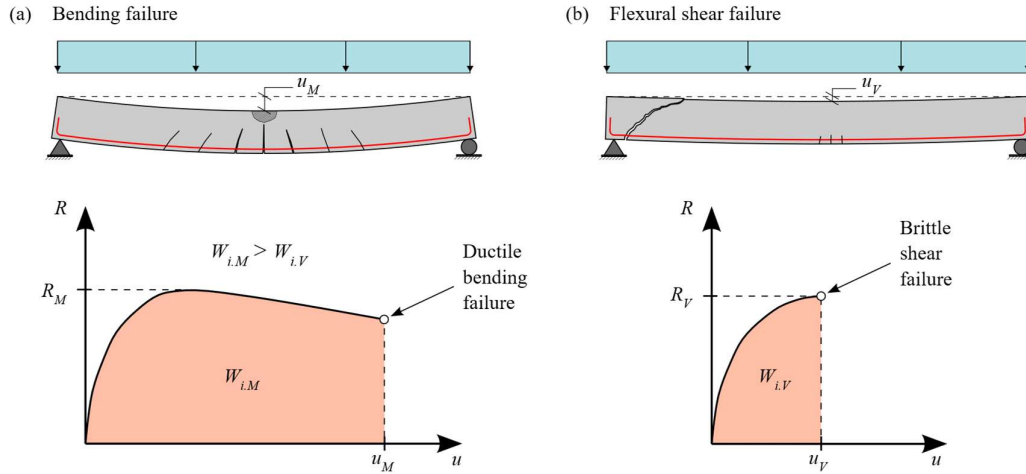


Figure 1. Schematic illustration of common failure mechanisms of a RC one-way slab subjected to a distributed blast load: (a) bending moment failure, (b) flexural shear failure. The shaded area under the resistance curve, $R(u)$, represents the internal work, W_i . Adapted from [5].

Thus, during the design of blast-loaded RC slabs, the designer must ensure that the total load capacity to shear failure is greater than the total load capacity to bending failure. This is a significant difference in comparison with statically loaded elements, for which it is usually not relevant which failure mode prevails, provided that the load capacity is greater than the applied load. However, in blast-loaded RC slabs the design shear demand is in practice a function of the bending resistance of the slab, rather than of the applied blast load. This implies that an overly conservative design regarding bending capacity may potentially increase the critical shear force, which may result in premature shear failure. Furthermore, predicting the dominant failure mode is a difficult task due to the uncertainties involved in the calculation of the bending and shear capacity. Sources of such uncertainties include the statistical variation of the material and geometrical properties as well as the uncertainties related to the resistance and blast load models.

Such uncertainties might lead to the false conclusion that the slab will fail under bending. Indeed, even though the *design*² shear resistance ($R_{V,d}$) may be found to be greater than the *design* bending resistance ($R_{M,d}$) for a given case, there still exists the possibility that the *real* shear resistance ($R_{V,real}$) is lower than the *real* bending resistance ($R_{M,real}$). The consequence of this, concerning the energy absorption capacity of the slab, is illustrated schematically in Figure 2, where the response of a RC one-way slab is represented by an idealised resistance function. In Figure 2, the *design* values of capacity indicate that the element will develop at ductile bending failure with deformation capacity u_R . Under such conditions, the element is designed to mobilize an internal work, $W_{i,d}$, achieved at a deformation u_{tot} . However, should the *real* shear resistance ($R_{V,real}$) be lower than the *real* bending resistance ($R_{M,real}$), the *real* energy that can be dissipated is represented by the area under the

² This article uses the adjective *design* to describe parameters or properties determined using code-specified nominal material strength, geometry, resistance models, and safety factors.

resistance function up to the deformation at shear failure, $u_{V,real}$. Hence, the real available internal work may be considerably lower than the intended internal work (i.e. $W_{i,real} \ll W_{i,d}$).

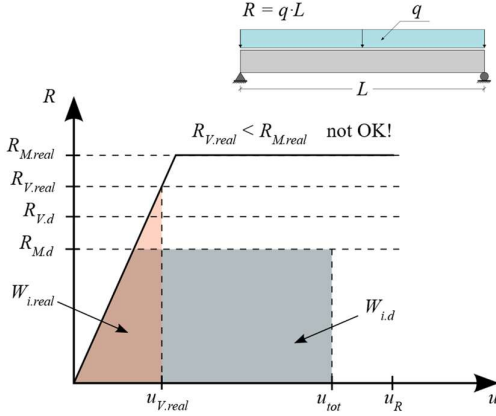


Figure 2. Idealised response of a RC slab. The area under the resistance curve up to the deformation $u_{V,Real}$ represents the real energy absorption capacity, $W_{i,real}$, when shear failure is the dominant failure mechanism.

Several research campaigns have shown that simply supported RC beams designed to fail under bending action when statically loaded may instead fail due to shear action if subjected to an extreme dynamic load. One example is the experimental research of blast-loaded RC beams carried out by Magnusson et al. [6]. The research program consisted of 11 beam types with constant geometry ($L/d \approx 12$) but with a unique combination of concrete class and reinforcement amount. In total, 49 beams were tested. 11 beams, one for each beam type, were tested statically. The remaining 38 beams were subjected to blast loading inside a shock tube. While all statically loaded beams failed under bending, some of the blast-loaded beams displayed a brittle shear failure. It was observed that the risk for shear failure increased for greater reinforcement amounts. Furthermore, beams that failed in shear exhibited limited deformation compared to the beams that failed in bending. This study showed that the failure mode of nominally identical beams may change from bending to a shear mechanism when loaded by a blast wave, which consequently decreases the energy absorption capacity of the beam.

Morales-Alonso et al. [7] conducted an experimental test on 12 square RC slabs ($L/d \approx 7$) supported at the corners. Six of the slabs were cast with normal strength concrete (NSC), while the other six were made of high strength concrete (HSC). The slabs had the same nominal geometry and reinforcement layout and were subjected to blast load. The theoretical failure mode of the slabs if statically loaded was bending action. The experimental results showed that of the six NSC slabs, one failed in bending and the remaining five failed in shear. Among the HSC slabs, two failed in bending, two in shear and two under a combined mode. The scatter of the failure modes, despite identical nominal geometrical and loading conditions, was attributed to the heterogeneity of the concrete material and irregularities in the geometry. This study showed that uncertainties in the materials and geometry may have an influence on the response of nominally identical blast-loaded elements.

Lozano and Makdesi [8] conducted an experimental research of plastic rotational capacity of impact-loaded simply supported RC beams with equal nominal geometry ($L/d \approx 12$) and reinforcement amount. The beams were subjected to combined impact and static load. Six of the beams were only tested statically under three-point bending. Six beams were loaded by the same mass falling from a height of 5.0 m. The impact load was not strong enough to cause failure of these beams. Instead, the impact-damaged beams were brought to failure under static loading after the impact test. Among the six impact-loaded beams, two developed a significant web shear crack during the impact test, while the remaining beams only developed flexural cracks. When these beams were subsequently tested under static load, five of them showed a ductile response. However, one

specimen (which had previously developed a web shear crack under impact loading) failed due to a brittle shear mechanism. The plastic rotational capacity of this element was around 20 % of the plastic rotational capacity of the other five specimens. This study also showed that dynamically loaded beams that are nominally identical and exposed to the same loading, may show different failure modes, likely influence by random variations in material properties, geometry or loading conditions.

The need to avoid brittle failure in favour of a ductile mechanism is not unique to blast-loaded elements. Indeed, this is one of the main requirements within seismic design of structures. To achieve this goal, the *capacity design* philosophy, originally developed primarily in New Zealand [9,10], has been implemented in several seismic design regulations, e.g. [11–13]. According to this approach, elements within the structures (usually beams) are selected and properly designed and detailed for high ductility and high energy absorption capacity. The shear demand in these dissipative elements is not determined directly from the applied load but instead is based on the design actions corresponding to the intended ductile mechanism amplified by an overstrength factor (“strong shear, weak bending”). This overstrength factor usually varies between 1.0 and 1.3 depending on the seismic class of the design. At the same time, the design action on the non-ductile members (usually columns) are determined by a suitable overstrength difference from the load bearing capacity of the dissipative elements (“strong column, weak beam”). However, even though this philosophy is well established within the context of seismic design, it is not commonly implemented for the design of blast-loaded structures. Furthermore, it is sometimes not clear what reliability level is associated with the code-specified overstrength factors or whether the same reliability level would apply for blast-loaded structures. Finally, the overstrength factors for “strong shear, weak bending” design are primarily based on two parameters related to the reinforcing steel: the difference between the real material strength and the code-specified nominal strength, and strain hardening. Other sources of uncertainties, such as geometrical uncertainties or model uncertainties, may not be included.

In this article, a probabilistic analysis was conducted to assess the effects of relevant uncertainties on the interaction between bending resistance and flexural shear resistance of slender RC (non-prestressed) one-way slabs subjected to blast loading. The sources of uncertainties considered in the study included the material and geometrical properties, and the models for resistance, blast load and strain rate effects. The intention was to quantify the probability of premature shear failure developing before bending failure. The resistance models in Eurocode 2 [14] were adopted. The overall aim of this work is to open avenues for academic discussion and further research concerning this topic, which, to the knowledge of the authors, has received minor attention in the literature. The analytical study included RC one-way slabs with and without shear reinforcement (stirrups). The reinforcement amount of the slabs was designed to achieve a given utilization of the plastic deformation capacity for different imposed blast loads. The assessment was performed using the conventional Monte Carlo (MC) method. The MC method has been extensively used in research efforts dealing with uncertainty and reliability of blast-loaded structures, e.g. [15–18]. A literature survey was conducted to select appropriate statistical parameters to define the relevant probability distributions.

The determination of the critical shear force for the studied cases was based on the concept of equivalent static load, which is defined as the equivalent distributed load at bending failure of the element. With this assumption, the shear demand in the element is estimated from the equivalent static load in similar fashion to statically loaded elements. However, it should be noted that the maximum dynamic shear force and the distribution of shear forces along a blast-loaded element may differ from those of a statically loaded element [19,20]. Nevertheless, the equivalent static load is widely used by engineers to determine the design shear demand, supported by design guidelines such as [2,3,21]. Therefore, in the interest of carrying out the study from the perspective of a commonly followed design methodology, the equivalent static load was adopted as the basis for calculating the design shear force.

2 Methodology

2.1 Overview

This article evaluates the likelihood of premature shear failure (rather than bending failure) in blast-loaded slender RC one-way slabs, both with and without shear reinforcement. The probabilistic assessment was performed analytically using the conventional Monte Carlo (MC) method. A non-linear single-degree-of-freedom (SDOF) model was implemented to predict the dynamic response. Moreover, the study adopted the models in Eurocode 2 for bending, deformation, and shear capacity, in combination with corresponding dynamic increase factors to account for strain rate effects.

The choice of the adopted models is justified by three main reasons. First, the relative simplicity of these models enables millions of iterations for over a hundred specimens using the MC method. Such a broad analysis would be much more resource-intensive with more advanced methods, such as non-linear finite element analysis (FEA). A simplified analysis with a wide scope can allow to identify the most important parameters that should receive special attention in future research work. Secondly, even though the resistance models in Eurocode 2 are originally intended for static and quasi-static loading, these models are today widely used in Europe for designing blast-loaded RC structures, supported by design guidelines such as [2,3]. These guidelines are largely based on the design approach in UFC 3-340-02 [21], which is perhaps the most widely used manual for design and analysis of structures exposed to explosion loads [22]. UFC 3-340-02 adopts the resistance models for shear and bending capacity in ACI 318 [23], which were also originally developed for quasi-static loading, modified by corresponding dynamic increase factors. Finally, one objective of the study is to conduct the probabilistic analysis from the perspective of a commonly used design methodology, which facilitates, if necessary, formulating a strategy for decreasing the risk of premature shear failure that can be directly implemented in the design approach.

During the design of blast-loaded RC one-way slabs, it is usually assumed that a bending plastic hinge can form in the structure, leading to higher deformation capacity and, thus, higher energy absorption capacity. However, for this assumption to be valid, premature brittle shear failure must be avoided before the ultimate moment capacity, M_R , is reached and the element has undergone the required plastic deformation. The shear capacity of the slab, V_R , must then be greater than the corresponding shear demand that arises when the slab has achieved its ultimate moment capacity, $V_E(M_R)$. That is, the shear force that the slab should be designed or checked for depends on the bending capacity of the element, as given by Eq. (1).

$$V_R > V_E(M_R) \quad (1)$$

Under the assumption that the load acting on a simply supported RC slab at failure could be represented by an equivalent uniformly distributed load, $V_E(M_R)$ can be calculated by Eq. (2), in which L is the free span of the slab and a_V is the distance from the support to the critical section. Assuming point supports, the distance a_V is given by Eq. (3) for members without stirrups ($a_{V,c}$) and Eq. (4) for members with stirrups ($a_{V,s}$), in which d is the effective depth, z is the internal lever arm, and ϕ is the strut inclination.

$$V_E(M_R) = \frac{8 \cdot M_R}{L^2} \cdot \left(\frac{L}{2} - a_V \right) \quad (2)$$

$$a_{V,c} = d \quad (3)$$

$$a_{V,s} = z \cdot \cos \phi \quad (4)$$

The design condition in Eq. (1) can be expressed more conveniently in terms of the resistance, R , defined as the total load acting on the slab at a given failure mode. That is, the resistance to shear failure, R_V , must be greater than the resistance to bending failure, R_M , as given by Eq. (5).

$$R_V > R_M \quad (5)$$

For the simply supported RC one-way slabs studied in this article, the resistance to bending failure was determined with Eq. (6). The resistance to shear failure was calculated according to Eq. (7).

$$R_M = 8 \cdot M_R / L \quad (6)$$

$$R_V = V_R \cdot L \cdot \left(\frac{L}{2} - a_V \right)^{-1} \quad (7)$$

The assessment in terms of the interaction between bending and shear resistance of the blast-loaded slabs was carried out by defining the limit state function G as the ratio between the resistance to shear failure and the resistance to bending failure, as given by Eq. (8). The variables R_V and R_M were treated as random variables due to the uncertainties associated with the material, geometry, resistance and blast load models, and strain rate effects. These uncertainties were described by probability density functions based on research studies published in the literature.

$$G = g(R_V, R_M) = R_V / R_M \quad (8)$$

The quantity G inherits a random nature from R_V and R_M . A value of G lower than 1.0 denotes a case in which the available shear resistance is lower than the bending resistance. This indicates that the (undesired) shear failure will likely govern the overall resistance of the slab. Conversely, a value of G greater than 1.0 represents a case in which the dominant failure mode is bending failure (preferred). The probability of premature shear failure, P_f , can then be expressed mathematically as

$$P_f = P\{G < 1\} \quad (9)$$

This study used the conventional MC method to solve Eq. (9). With this method, the probability of premature shear failure was calculated with Eq. (10), in which n is the number of times the criteria $G < 1$ is fulfilled and N is the total number of simulations. The general procedure followed is outlined in Figure 3. The number of simulations for each evaluated specimen was set to $N = 10^6$.

$$P_f \approx \frac{n}{N} \quad (10)$$

Note that the variables R_M , R_V , G , and P_f were defined in general terms in this section. However, where relevant, the subscripts s and e were added to distinguish between elements with and without stirrups, respectively. These are the same subscripts use to distinguish between shear capacity with and without stirrups in Eurocode 2.

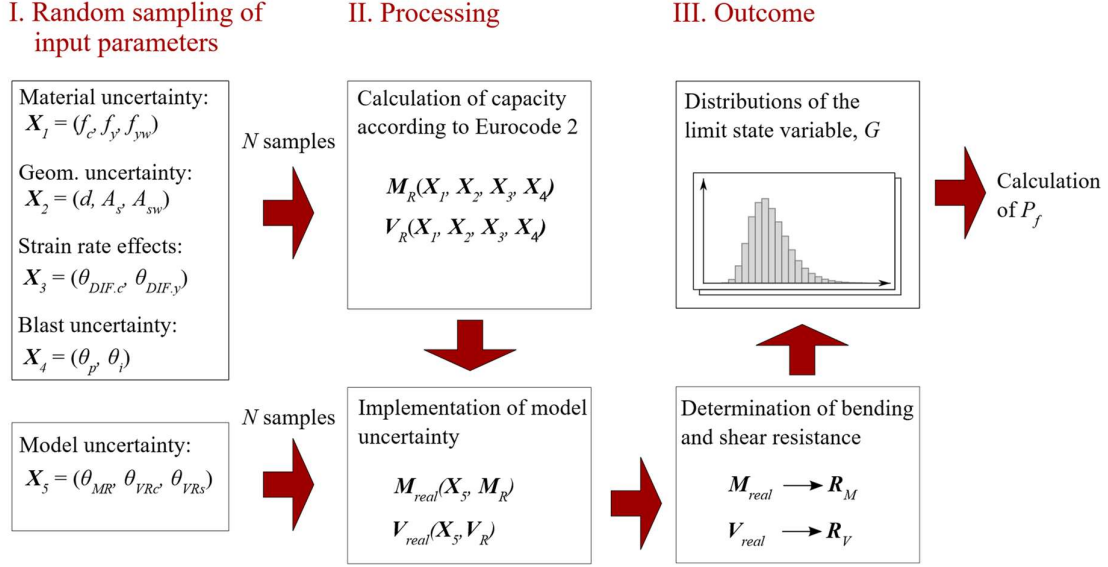


Figure 3. Flow diagram of the implementation of the Monte Carlo method. The variables and uncertainties referred to in the diagram are defined in the following sections.

2.2 Resistance models in Eurocode 2

The moment capacity of the RC slabs was calculated according to the simplified rectangular stress block in Eurocode 2 using Eq. (11). The parameters A_s and f_{dy} are the area and dynamic yield strength of the longitudinal reinforcement bars. The determination of the dynamic strength of materials is discussed in Section 2.3. The factors λ and η are equal to 0.8 and 1.0 for a characteristic compressive strength, f_{ck} , less than or equal to 50 MPa. An idealised stress-strain diagram with a horizontal top branch was assumed for the reinforcing steel. The contribution of strain hardening to the moment capacity is implicitly considered in the model uncertainty described in Section 4.2. Any contribution from compression reinforcement was disregarded. The depth of the compressive zone, x_u , was determined with Eq. (12), in which f_{dc} is the dynamic compressive strength of concrete and b is the width of the cross section.

$$M_R = f_{dy} \cdot A_s \cdot (d - 0.5 \cdot \lambda \cdot x_u) \quad (11)$$

$$x_u = f_{dy} \cdot A_s / (\eta \cdot f_{dc} \cdot b \cdot \lambda) \quad (12)$$

The capacity to shear failure without stirrups, $V_{R,c}$, was determined with the empirical expression given by Eq. (13), in which $\rho_l (= \min\{A_s/bd, 0.02\})$ is the reinforcement ratio. The parameter k , calculated using Eq. (14), accounts for the size effect. The shear capacity with vertical stirrups was calculated using Eq. (15), in which A_{sw} , f_{dyw} and s are the area, dynamic yield strength and spacing of the stirrups, respectively. This capacity is based on a strut and tie model with strut inclination ϕ . The maximum allowed inclination was used, i.e. $\cot\phi = 2.5$.

$$V_{R,c} = \max \begin{cases} 0.18 \cdot k \cdot (100\rho_l \cdot f_{dc})^{1/3} \cdot b \cdot d \\ 0.035 \cdot k^{1.5} \cdot \sqrt{f_{dc}} \cdot b \cdot d \end{cases} \quad (13)$$

$$k = \min \left(1 + \sqrt{\frac{200}{d}}, 2 \right) \quad (14)$$

$$V_{R.s} = \frac{A_{sw}}{s} \cdot (d - 0.5 \cdot \lambda \cdot x_u) \cdot f_{dyw} \cdot \cot \phi \quad (15)$$

The minimum amount of shear reinforcement was calculated using Eq. (16), in which $s_{max} = 0.75 \cdot d$ is the maximum longitudinal spacing between stirrups.

$$\rho_{w.min} = \max \left\{ \frac{0.08 \cdot \sqrt{f_{ck}}}{f_{ywk}}, \frac{A_{sw}}{(s_{max} \cdot b)} \right\} \quad (16)$$

The plastic rotational capacity at the support, α_s , was determined using Eq. (17), in which k_λ is a correction factor that accounts for the shear slenderness of the beam. The parameter α_{pl} is the base allowable rotation, which depends on the reinforcement class, concrete class and the ratio x_u/d . Figure 4 gives α_{pl} for reinforcement class C and concrete class $\leq C50/60$. Finally, the total deformation capacity of the slab, u_R , is calculated with Eq. (18), in which u_e is the maximum elastic deformation.

$$\alpha_s = \frac{k_\lambda}{2} \cdot \alpha_{pl} \quad (17)$$

$$u_R = \frac{\alpha_s \cdot L}{2} + u_e \quad (18)$$

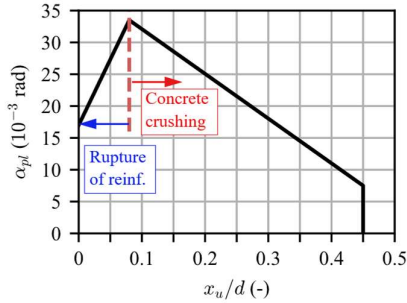


Figure 4. Base allowable rotation α_{pl} for concrete class $\leq C50/60$ and reinforcement class C according to Eurocode 2 [14].

It should be noted that Figure 4 specifies the allowable rotation for $x_u/d \leq 0.45$. According to Eurocode 2, no plastic rotational capacity can be utilised for $x_u/d > 0.45$. This condition imposes an upper limit on the amount of longitudinal reinforcement, which is given by Eq. (19).

$$\rho_{l.max} = 0.36 \cdot \frac{f_{dc}}{f_{dy}} \quad (19)$$

2.3 Strain rate effects

2.3.1 Strain rate effects under bending action

It is commonly accepted that the apparent strength of materials subjected to high strain rates can be significantly greater than the corresponding material properties under quasi-static loading. Therefore, when

using resistance models developed for quasi-static load for designing blast-loaded structures, complementary dynamic increase factors (DIFs) are normally used to account for the influence of high strain rate on material strengths [3,21]. The DIF is defined as the ratio of dynamic to quasi-static strength:

$$\text{DIF} = f_{\text{dynamic}}/f_{\text{static}} \quad (20)$$

In this article, the enhancement of compressive strength of concrete under bending was estimated by Eq. (21) [24], in which $\alpha_c = (5 + 0.75 \cdot f_{cu})^{-1}$, $\log(\gamma) = 6.156 \cdot \alpha_c - 0.492$, $\dot{\epsilon}_c$ is the compressive strain rate in the concrete, f_{cu} is the static cube compressive strength in MPa, and $\dot{\epsilon}_{cs} = 30 \times 10^{-4} \text{ s}^{-1}$.

$$\text{DIF}_{\text{c.M}} = \begin{cases} \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{cs}} \right)^{1.026\alpha_c}, & \dot{\epsilon}_c \leq 30 \text{ s}^{-1} \\ \gamma \left(\frac{\dot{\epsilon}_c}{\dot{\epsilon}_{cs}} \right)^{1/3}, & \dot{\epsilon}_c > 30 \text{ s}^{-1} \end{cases} \quad (21)$$

For estimation of the dynamic increase of the yield strength in the longitudinal reinforcement bars, the relationship proposed by Malvar and Crawford [25], given by Eq. (22), was used. Here, $\alpha_y = 0.074 - 0.04f_y/414$, $\dot{\epsilon}_y$ is the strain rate of the reinforcement bar, and f_y is the yield strength of the reinforcement bar in MPa.

$$\text{DIF}_{\text{y.M}} = \left(\frac{\dot{\epsilon}_y}{10^{-4} \text{ s}^{-1}} \right)^{\alpha_y} \quad (22)$$

The strain rate under bending action was estimated using Eq. (23) and (24) for concrete and reinforcing steel [21], in which t_e is the time to reach the ultimate bending resistance. The time t_e was determined with the single-degree-of-freedom (SDOF) model described in Section 0.

$$\dot{\epsilon}_c = 0.002/t_e \quad (23)$$

$$\dot{\epsilon}_y = f_{dy}/(E_s t_e) \quad (24)$$

Clearly, the dynamic increase factors for bending action according to Eq. (21) and Eq. (22) depend on the material and geometrical properties of the specimen as well as on the characteristics of the blast load. For that reason, the values of $\text{DIF}_{\text{c.M}}$ and $\text{DIF}_{\text{y.M}}$ varied for the different specimens studied in this article. The values used during the *design* of the reinforcement amount of the specimens were in the order of 1.34 for $\text{DIF}_{\text{c.M}}$ and 1.24 for $\text{DIF}_{\text{y.M}}$.

2.3.2 Strain rate effects under shear action

While the effects of strain rate on the compressive and tensile strength of concrete and reinforcing steel have been extensively researched and are well understood, the current knowledge about the increase of the resistance to flexural shear failure in slender RC beams due to high loading rate is more inconclusive. Much of the research about dynamic shear resistance of RC members has focused on three areas: i) shear resistance in columns and beams subjected to impact loading, e.g. [26–29]; ii) shear resistance in deep and short beams under dynamic loading conditions, e.g. [30,31]; and iii) dynamic direct shear loading, e.g. [32,33]. Some of these works have suggested different strategies to consider the increase of shear resistance, including semi-empirical equations for calculating either the dynamic increase factor or the dynamic shear resistance directly. However, there are several challenges that make it difficult to correctly consider the results of these works to

the dynamic response of the blast-loaded slender RC slabs evaluated in this article. First, such studies are mostly focused on punching shear, shear compression (in deep beams) and direct shear. These mechanisms are different to the type of shear failure (flexural shear) relevant for the specimens in this study. Second, apart from direct shear applications, many of those studies treated the increase of shear resistance as a property of the structural element, rather than an inherent material property, and thus it is dependent on the geometry of the specimens, resistance models used, and the loading conditions. This means that results from these studies are not directly applicable to the conditions in this article. Finally, a key issue is the determination of a value of strain rate of the materials for calculating shear capacity at the critical section. Some studies have provided empirical relationships between the loading rate and an effective strain rate [27,31]. However, their applicability to the RC members studied here is not guaranteed.

While several experimental works dealing with blast-loaded RC elements have been published, few compare the experimental static capacity to the experimental dynamic shear capacity. A notable contribution is that of Magnusson et al. [6], who provided the ratio of ultimate dynamic load to ultimate static load of RC beams with shear reinforcement subjected to blast load based on experimental research. All beams tested under static loading developed a flexural failure. In contrast, some of the blast-loaded beams (nominally identical to their statically loaded counterpart) failed in shear. Among the beams made with high-strength concrete that developed shear failure under blast loading, this ratio was in average equal to 1.35. However, in beams with normal strength concrete, the ratio was equal to 1.04, which was attributed to the static shear resistance being close to the static flexural strength. This indicates that the increase in shear resistance can be relatively low, compared to the increase in flexural strength, and much lower than the increase reported in e.g. [30,31]. This implies that there is a potentially significant uncertainty involved in the estimation of the dynamic increase of the shear resistance which has yet to be characterised, to the knowledge of the authors.

Accordingly, dynamic shear resistance of slender RC beams remains an important knowledge gap. However, investigating this topic is beyond the scope of this article. Instead, the article utilizes the tabulated values of DIFs prescribed by the well-known design guideline UFC 3-340-02 [21]. The recommended values of DIFs for shear action are more conservative than those for bending, motivated by the need to prevent brittle shear failure and the uncertainty surrounding dynamic shear resistance. The values of DIFs for concrete ($DIF_{c,v}$) and reinforcing steel ($DIF_{y,v}$) under shear action for closed design range in UFC 3-340-02 were adopted:

$$DIF_{c,v} = 1.0 \quad (25)$$

$$DIF_{y,v} = 1.1 \quad (26)$$

Furthermore, the DIFs for shear action were kept constant throughout the entire analysis. This implies that the shear capacity may be underestimated for some situations. This approximation was deemed to be acceptable given the current state-of-the-art and is here viewed as an additional safety factor.

2.4 Dynamic analysis with a SDOF model.

The dynamic response of the RC slabs was predicted with a single-degree-of-freedom (SDOF) model, as shown in Figure 5. This was necessary to determine the maximum deformation due to the blast loading and the strain rate in the materials. The RC slabs were transformed into a SDOF system with the help of the load-mass transformation factor κ_{MF} , which is based on the conservation of energy of the system [1]. The equation of motion of the SDOF system is given by Eq. (27), in which m is the mass of the RC slab, $u(t)$ and $\ddot{u}(t)$ are the midspan deflection and acceleration, and $R(u(t))$ is the idealised elastoplastic resistance function in Figure 5. The slab was assumed to behave elastically until the maximum bending resistance was reached at u_e . The elastic stiffness is given by Eq. (28), in which E_c is the module of elasticity of concrete and I_{cr} is the moment

of inertial of the cracked cross section. The equation of motion was solved numerically with the Central Difference Method.

$$\kappa_{MF} \cdot m \cdot \ddot{u}(t) + R(u(t)) = b \cdot L \cdot p(t) = F(t) \quad (27)$$

$$K_e = 384 \cdot E_c \cdot I_{cr} / L^4 \quad (28)$$

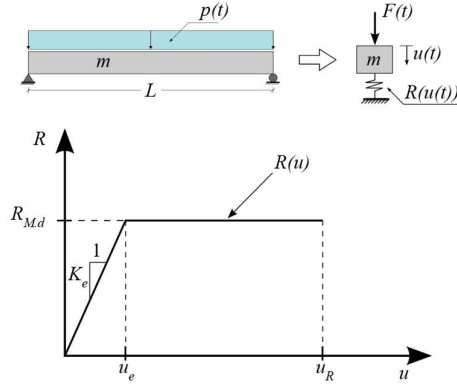


Figure 5. Schematic representation of the equivalent SDOF system and idealised resistance function of a RC slab.

3 Case study

The case study examined *strips* of slender RC one-way slabs subjected to a uniformly distributed blast load. The strips were simply supported over a single span and had unitary width ($b = 1$ m). Six slab types, with a unique combination of free span (L) and height (h) as given in Table 1, were studied. All geometries were evaluated both with and without shear reinforcement. In total, 244 slabs were studied. The chosen dimensions were meant to represent typical values for conventional concrete buildings and civil defence shelters. In agreement with the definition of *slender beams* in [34] and [35], all specimens fulfilled $L/d > 9.6$. Concrete class C35/45 ($f_{ck} = 35$ MPa) and reinforcement grade B500C ($f_{yk} = 500$ MPa) were adopted. The slabs were reinforced with longitudinal bars of diameter $\varnothing 16$ mm and, where applicable, stirrups of diameter $\varnothing 8$ mm (four legs per unit width). The amount of shear reinforcement (where applicable) was set to the minimum required amount given by Eq. (16). The concrete cover, c , was set at 40 mm, including stirrups and transversal reinforcement. A schematic illustration of the geometry of the slabs, loading condition, and reinforcement layout appear in Figure 6.

Table 1. Geometry of the studied slab types. For all specimens, $b = 1.0$ m.

Slab type	L (mm)	h (mm)	d (mm)	L/d (-)	s^* (mm)
1	3000	300	252	11.9	189
2	4000	300	252	15.9	189
3	4000	400	352	11.4	212
4	5000	300	252	19.8	189
5	5000	400	352	14.2	212
6	5000	500	452	11.1	212

* Longitudinal spacing of stirrups in members with shear reinforcement.

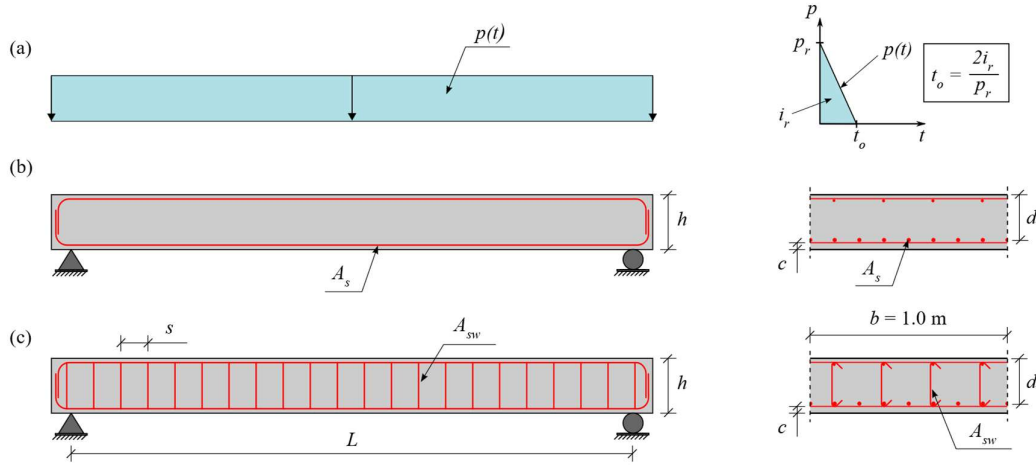


Figure 6. Geometry and loading conditions of case study: (a) uniformly distributed blast load, (b) geometry and reinforcement layout of the slabs without stirrups, (c) geometry and reinforcement layout of the slabs with stirrups.

For each slab type in Table 1, several specimens were generated by varying their reinforcement amounts. To that end, the amount of longitudinal reinforcement was calculated under the assumption that each slab type was subjected to the blast loads described in Table 2. The design bomb used for designing civil shelters in Sweden [36] was adopted as the reference load (L1). This design bomb consists of a spherical free-air burst of 125 kg TNT at 5.0 m from the target structure. The other blast loads were generated by varying either the

charge mass, W , or the stand-off distance, R . The reflected overpressure (p_r) and reflected impulse (i_r) were determined according to the equations by Kingery and Bulmash [37,38], which also provide the basis for the air blast calculations in UFC 3-340-02 and ConWep [39]. The blast load was idealised to a triangular pressure pulse with zero rise time as given in Figure 7. The effect of the self-weight of the slab was disregarded.

The slabs were designed so that the maximum deformation needed to dissipate the energy imposed by the load, u_{tot} , is both greater than the elastic limit, u_e , and lower than the total deformation capacity, u_R . For each blast load, three values of the utilization of the plastic deformation capacity ($\chi_{u,pl}$) were sought after: 0.2, 0.5, and 0.8. The utilization of the plastic deformation capacity is given by Eq. (29). The deformation u_{tot} was determined with the SDOF model described in Section 2.4, while u_R was calculated as described in Section 2.2. The shear resistance of the slabs was not verified at the design stage. Consequently, some of the specimens had a utilization of shear capacity *at design* that exceeds 1.0. Whether such beams would be likely to fail under shear action was investigated at a later stage with the Monte Carlo method.

$$\chi_{u,pl} = \frac{u_{tot} - u_e}{u_R - u_e} \quad (29)$$

Table 2. Design blast loads used in this study.

Load	W (kg)	R (m)	p_r (MPa)	i_r (kPa·s)
L1*	125	5	5.0	2.8
L2	125	6	2.9	2.2
L3	125	7	1.9	1.8
L4	125	8	1.2	1.5
L5	125	9	0.9	1.3
L6	150	5	6.0	3.2
L7	160	5	6.3	3.4
L8	180	5	7.1	3.7
L9	200	5	7.8	4.0

* Design bomb for design of civil shelters in Sweden [36]

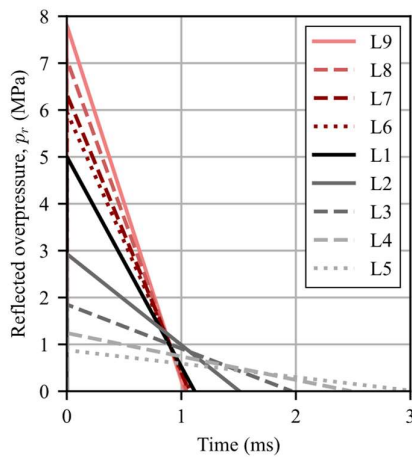


Figure 7. Overpressure-time history of the adopted design blast loads.

During the calculation of reinforcement amounts, the characteristic values of the material properties and the recommended partial factors for materials for accidental design situations in Eurocode 2 ($\gamma_c = 1.2$ for concrete and $\gamma_s = 1.0$ for reinforcing steel) were used. The outcome of Eq. (13) was also divided by $\gamma_c = 1.2$ in this stage. It should be noted that the characteristic values and partial factors for materials were used only to calculate reinforcement amounts and not during the MC simulations.

The amount of longitudinal reinforcement for all studied specimens is given in Table 3. It should be noted that not all combinations of slab type, blast load, and target utilization of the plastic deformation capacity were possible, as the required reinforcement amount would either exceed the maximum allowed amount given by Eq. (19) or be less than the minimum amount prescribed by Eurocode 2. In those cases, wherever possible, another value of the utilization of plastic deformation capacity was accepted. The same reinforcement amount in Table 3 applies for both specimens with and without stirrups. The maximum allowed support rotation for these specimens according to the provisions in Eurocode 2 ranged between 1.0 and 2.6 degrees. These values are comparable to the ultimate support rotation permitted by UFC 3-340-02 for members without shear reinforcement: 2 degrees. It should be noted, though, that UFC 3-340-02 allows for rotations up to 6 degrees in members with shear reinforcement without lateral restraint at the supports.

Table 3. Amount of longitudinal reinforcement, ρ_l , of the studied specimens as a function of the blast load, slab type and utilization of the plastic deformation capacity, $\chi_{u,pl}$. The dash symbol (-) indicates that no design was possible according to the criteria adopted in the study. The underscored values indicate deviation of $\chi_{u,pl}$ from the initially desired target. The parameter n gives the number of specimens for each slab type.

Load	Slab type 1 n = 22		Slab type 2 n = 20		Slab type 3 n = 23		Slab type 4 n = 16		Slab type 5 n = 20		Slab type 6 n = 21	
	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)
L1	<u>0.35</u>	2.16	<u>0.45</u>	2.26	0.20	0.95	-	-	0.20	1.02	0.20	0.54
L1	0.50	1.23	0.50	1.53	0.50	0.50	<u>0.52</u>	1.99	0.50	0.56	0.50	0.33
L1	0.80	0.72	0.80	0.87	0.80	0.36	0.80	1.01	0.80	0.39	0.80	0.25
L2	<u>0.10</u>	1.51	<u>0.10</u>	1.61	-	-	<u>0.10</u>	1.70	-	-	-	-
L2	0.20	1.16	0.20	1.29	0.20	0.55	0.20	1.40	0.20	0.59	0.20	0.35
L2	0.50	0.61	0.50	0.71	0.50	0.33	0.50	0.80	0.50	0.35	0.50	0.23
L2	0.80	0.41	0.80	0.47	0.80	0.25	0.80	0.53	0.80	0.27	0.80	0.17
L3	0.20	0.73	0.20	0.80	0.20	0.38	0.20	0.85	0.20	0.40	0.20	0.26
L3	0.50	0.40	0.50	0.45	0.50	0.25	0.50	0.50	0.50	0.26	0.50	0.17
L3	0.80	0.31	0.80	0.34	0.80	0.19	0.80	0.36	0.80	0.20	0.80	0.13
L4	0.20	0.50	0.20	0.55	0.20	0.29	0.20	0.60	-	-	-	-
L4	0.50	0.31	0.50	0.34	0.50	0.19	0.50	0.36	-	-	-	-
L4	0.80	0.24	0.80	0.26	0.80	0.14	0.80	0.28	-	-	-	-
L5	0.20	0.38	0.20	0.41	0.20	0.23	0.20	0.44	-	-	-	-
L5	0.50	0.25	0.50	0.27	0.50	0.15	0.50	0.29	-	-	-	-
L5	0.80	0.19	0.80	0.21	-	-	0.80	0.23	-	-	-	-
L6	<u>0.68</u>	1.73	<u>0.81</u>	1.70	0.20	1.39	-	-	0.20	1.52	0.20	0.75
L6	0.80	1.17	<u>0.90</u>	1.25	0.50	0.71	-	-	0.50	0.80	0.50	0.40
L6	<u>0.95</u>	0.92	<u>0.95</u>	1.14	0.80	0.46	<u>0.94</u>	1.60	0.80	0.52	0.80	0.31
L7	<u>0.81</u>	1.70	<u>0.95</u>	1.60	0.20	1.68	-	-	0.20	1.84	0.20	0.86
L7	<u>0.89</u>	1.23	-	-	0.50	0.82	-	-	0.50	0.92	0.50	0.45
L7	<u>0.95</u>	1.11	-	-	0.80	0.52	-	-	0.80	0.59	0.80	0.33

Table 3. Continued

Load	Slab type 1 n = 22		Slab type 2 n = 20		Slab type 3 n = 23		Slab type 4 n = 16		Slab type 5 n = 20		Slab type 6 n = 21	
	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)	$\chi_{u,pl}$ (-)	ρ_l (%)
L8	-	-	-	-	<u>0.28</u>	2.20	-	-	<u>0.35</u>	2.24	0.20	1.06
L8	-	-	-	-	0.50	1.08	-	-	0.50	1.24	0.50	0.56
L8	-	-	-	-	0.80	0.66	-	-	0.80	0.75	0.80	0.38
L9	-	-	-	-	-	-	-	-	-	-	0.20	1.35
L9	-	-	-	-	-	-	-	-	<u>0.55</u>	1.90	0.50	0.68
L9	-	-	-	-	-	-	-	-	0.80	0.97	0.80	0.44

4 Uncertainties

4.1 Material and geometrical properties

Many statistical models for material and geometrical properties can be found in the literature, compiled in such works as [40–43]. Regarding the material properties, this study included the statistical variation of both the compressive strength of concrete and the yield strength of the reinforcing steel.

The uncertainties associated with the concrete cover, c , and the height of the slab, h , were considered jointly through the effective depth ($d = h - c$) [42]. The variability of the effective depth is given by Eq. (30), in which d_N is the nominal value of the effective depth and Δd is the error in effective depth. The width of the cross section and the span of the one-way slabs were included as constant values. The uncertainty related to the cross-sectional area of the reinforcement bars was assumed to be implicitly considered in the variability of the yield strength, as the latter is defined based on the nominal area of the reinforcement bars. A summary of the statistical characteristics of relevant input parameters appears in Table 4.

$$d = d_N + \Delta d \quad (30)$$

Table 4. Statistical characteristics of relevant material and geometrical parameters.

Parameter	Mean, μ	CoV*	Distribution	Source
Compressive strength, f_c	39.9 MPa	0.186	Lognormal	[44], [43]
Yield strength, f_y and f_{yw}	560 MPa	0.054	Normal	[42]
Error in effective depth, Δd	10 mm	1.0	Normal	[42]

* The coefficient of variation, CoV , is defined as σ/μ , in which σ is the standard deviation.

4.2 Model uncertainty

The model uncertainty is represented by the random variable θ , which accounts for the influence of random effects that are neglected in the theoretical models and simplifications in the mathematical formulations [45]. The relationship between the theoretical value and the experimentally determined value is described by the multiplicative relationship given by Eq. (31).

$$\text{Real value} = \theta \times \text{Theoretical value} \quad (31)$$

This work considered the uncertainties related to the resistance models in Eurocode 2 for reinforced concrete elements subjected to bending (moment capacity, θ_{MR}) and shear (shear capacity of members without and with shear reinforcement, θ_{VRc} and θ_{VRs}). Furthermore, the uncertainties associated with the estimation of the strain rate effects under bending action were considered. These included the error in the determination of the dynamic increase factors for concrete compressive strength, $\theta_{DIF,c}$, and yield strength, $\theta_{DIF,y}$, as well as the uncertainties in the calculation of the reflected overpressure, θ_p , and impulse, θ_i , caused by the explosive charge. It was assumed that the uncertainties of the different models could be treated independently. The choice of the distributions of the uncertain parameters was primarily based on published research works. The exceptions were the distributions of the uncertainties associated to θ_{VRc} and $\theta_{DIF,c}$, which were determined in this study based on comparison against experimental databases. A summary of the chosen parameters representing the model uncertainties appears in Table 5. The uncertainty distributions related to the resistance models are shown in Figure 8.

Table 5. Statistical characteristics of relevant model uncertainties.

Parameter	Mean, μ	CoV	Distribution	Source
Moment capacity, θ_{MR}	1.075	0.075	Lognormal	[46]
Shear capacity without stirrups, θ_{VRc}	--	--	Burr XII*	Section 4.2
Shear capacity with stirrups, θ_{VRs}	1.47	0.264	Lognormal	[47]
DIF for compressive strength of concrete in bending, $\theta_{DIF,c}$	1.047	0.129	Lognormal	Section 4.2
DIF for yield strength of longitudinal reinforcement bars, $\theta_{DIF,y}$	1.0	0.01	Normal	[48]
Blast overpressure**, θ_p	1.06-0.005 $\cdot Z$	0.01175-0.0125 $\cdot Z$	Lognormal	[49]
Blast impulse**, θ_i	0.99	0.222-0.054 $\cdot Z$	Lognormal	[49]

* Defined by Eq. (32) with parameters $c = 14.495$, $k = 0.411$, $S = 0.949$.

** $Z = R/W^{1/3}$ (m/kg^{1/3}) is the scaled distance, R is the distance to the charge, and W is the charge mass.

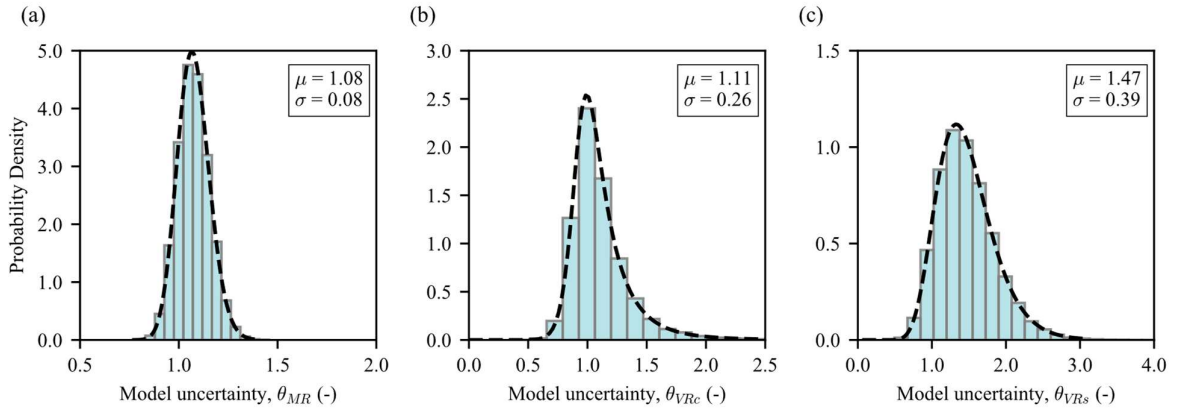


Figure 8. Distribution of the randomly drawn values of model uncertainty for: (a) moment capacity θ_{MR} , (b) shear capacity without stirrups θ_{VRc} , (c) shear capacity with stirrups θ_{VRs} .

The defining parameters of the probabilistic distribution of the uncertainty related to moment capacity were taken from [46]. This uncertainty is represented by a lognormal distribution with mean $\mu_{\theta,MR} = 1.075$ and coefficient of variation $CoV_{\theta,MR} = 0.075$, which were based on the analysis of an experimental database with 109 RC beams reported in [50]. These values are consistent with those reported by other authors, see e.g., [51].

The model uncertainty related to the shear capacity with shear reinforcement was characterised by a lognormal distribution with $\mu_{\theta,VRs} = 1.47$ and $CoV_{\theta,VRs} = 0.264$, as reported by Cladera et al. [47], based on a comparison with the ACI-DafStb database consisting of 170 RC beams with stirrups [35].

Among the different failure modes included in this study, prediction of shear resistance of RC members without shear reinforcement is arguably the most challenging due to the lack of a physical model that fully describes the mechanism of shear transfer [52]. The current Eurocode 2 uses an empirical model based on regression analysis of experimental data. The uncertainty related to this model, θ_{VRc} , has been studied in e.g. [46,47,53–55]. Cladera and Mari [53] reported that the model uncertainty θ_{VRc} was influenced by the effective height of the cross-section and amount of longitudinal reinforcement. It was observed that the model tends to overpredict the resistance of members with greater cross-section heights ($d \geq 900$ mm) or low amounts of longitudinal

reinforcement ($\rho \leq 1\%$). In contrast, the model was observed to underestimate the resistance of members with small cross-section heights ($d \leq 100$ mm) or with reinforcement ratios above the upper limit ($\rho > 2\%$). Consequently, a given value of model uncertainty θ_{VRc} should preferably only be used for members with properties similar to those of the specimens used to characterise the uncertainty. Hence, in this study, the statistical characteristics and suitable probability distribution for θ_{VRc} for the range of slab specimens of interest were estimated by comparison to the experimental results in the ACI-DAfStb database for slender RC beams [56]. The dataset was filtered to consider only members in the range: $100 \text{ mm} < d \leq 900 \text{ mm}$, $\rho \leq 2\%$ and $f_c \leq 50 \text{ MPa}$. Of 892 specimens, 456 remained after filtering the database. Goodness-of-fit tests were performed on several distributions, using probability plots and the Akaike information criterion (AIC) [57]. The distributions tested were: normal, lognormal, generalised extreme value (GEV), Burr XII [58], gamma, and beta. The three-parameter Burr XII distribution [58], given by Eq. (32), was found to provide the best fit and was therefore used here. The fitted parameters for this distribution are $c = 14.495$, $k = 0.411$ and $S = 0.949$.

$$f(x) = \frac{k \cdot c \cdot \left(\frac{x}{S}\right)^{c-1}}{S \cdot \left[1 + \left(\frac{x}{S}\right)^c\right]^{k+1}} \quad (32)$$

The statistical characteristics of the model uncertainty associated with the dynamic increase factor of compressive strength of concrete, $\theta_{DIF,c}$, was evaluated on the database of compression tests at high strain rate in [59]. The database was filtered to only consider specimens with $20 \text{ MPa} < f_c \leq 50 \text{ MPa}$. The two-parameter lognormal distribution with mean 1.047 and standard deviation 0.135 was found to provide satisfactory fit.

5 Results and discussion

5.1 Analysis of selected reference specimens

This section showcases the statical analysis conducted at a specimen level. The results presented are for two reference slab strips with equal *design* resistance to shear and bending failure. That is, $R_{M,d} = R_{V,d}$. The height and span of the reference slabs are $h = 300$ mm and $L = 3.0$ m. The reference specimen without stirrups has reinforcement ratio $\rho_l = 0.4$ % and was designed for blast load 2 in Table 2. The specimen with stirrups has reinforcement ratio $\rho_l = 1.1$ % and was subjected to blast load 7.

Figure 9 gives the outcome of the MC simulations for the reference specimen without stirrups. The probabilistic distributions of the resistance to shear ($R_{V,c}$) and bending ($R_{M,c}$) appear in Figure 9(a). The distribution of $R_{M,c}$ is nearly symmetrical, although it has a slightly fatter right tail. In contrast, the distribution of $R_{V,c}$ is positive-skewed, meaning that the mass of the distribution is concentrated on the left side while the right tail is longer. This implies that values of shear resistance which are much larger than the mean value are possible, although their likelihood decrease rapidly further away from the mean. This is a direct consequence of the uncertainty related to the resistance models. In general, the model of shear resistance is more uncertain, and therefore greater deviation from the predicted value is expected. The uncertainty distribution of the limit state variable (G_c) is given by Figure 9(b). A mean value greater than 1.0 indicates that the expected failure mode is bending. It can be seen that the skewness of the distribution of the model uncertainty θ_{VRc} is inherited by the distribution of G_c . The mean value and standard deviation of G_c was 1.2 and 0.31, respectively. For this specimen, the calculated probability of premature shear failure occurring before bending failure was $P_{f,c} = 26$ %. Hence, the results show that the studied reference specimen, originally designed to reach bending capacity and shear capacity at the same load, is more likely to fail under bending when subjected to blast loading. However, depending on the risk tolerability for a given scenario, the estimated probability of premature shear failure may still be considered unacceptable, as the consequences of shear failure are likely to be more severe.

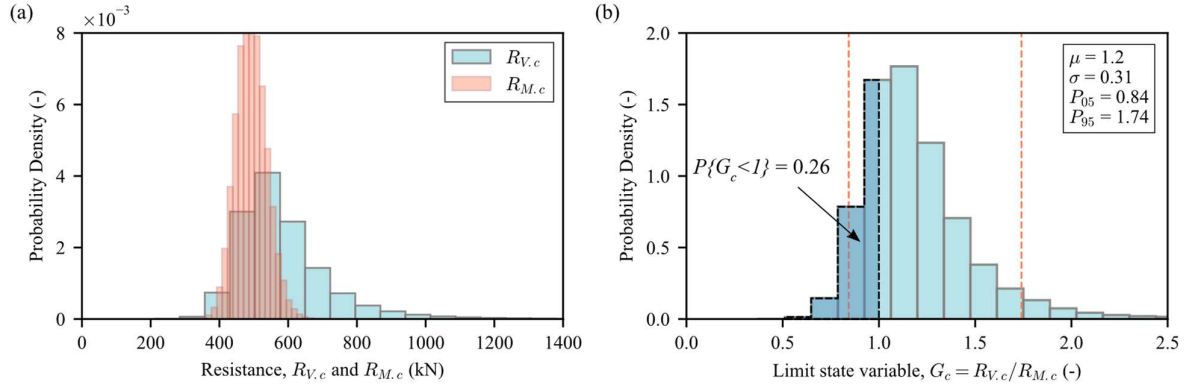


Figure 9. Outcome of the MC simulations for the reference specimen without stirrups ($h = 300$ mm, $L = 3.0$ m): (a) uncertainty distribution of $R_{V,c}$ and $R_{M,c}$, (b) uncertainty distribution of the limit state variable G_c .

The uncertainty distribution of the limit state variable, G_s , and the probabilistic distribution of the resistance to shear ($R_{V,s}$) and bending ($R_{M,s}$) for the reference slab with shear reinforcement are presented in Figure 10. As with the slab without stirrups, the positive skewness of the lognormal distribution of the model uncertainty for shear capacity, θ_{VRs} , had an evident impact on the shape of the distribution of G_s . The mean value and standard deviation of G_s are equal to 1.52 and 0.43. The estimated probability of premature shear failure was $P_{f,s} = 9$ %, which is lower than the corresponding probability calculated for the reference slab without stirrups. Consequently, members with shear reinforcement appear to have a more favourable response regarding the

preferred failure mode. This agrees with the fact that the model uncertainty for shear resistance is more favourable in members with stirrups, as shown in Figure 8.

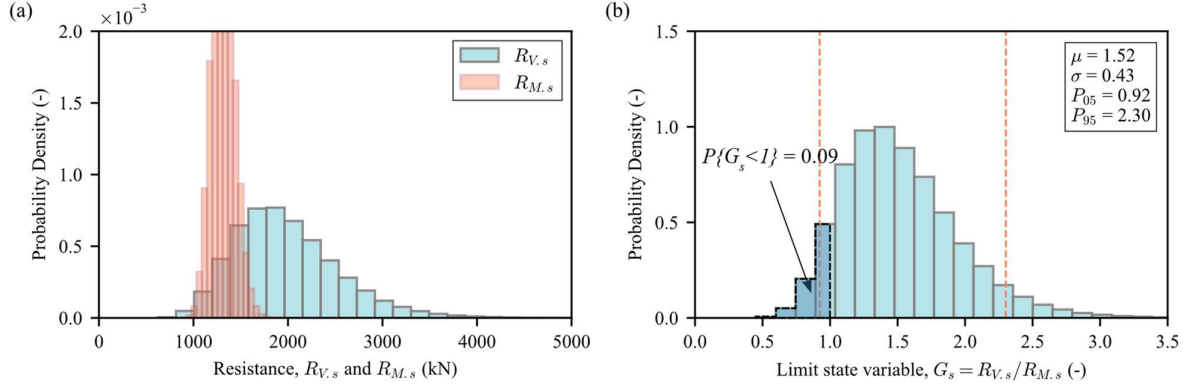


Figure 10. Outcome of the MC simulations for the reference specimen with stirrups ($h = 300$ mm, $L = 3.0$ m): (a) uncertainty distribution of $R_{V,s}$ and $R_{M,s}$, (b) uncertainty distribution of the limit state variable G_s .

It should be noted that any value of P_f calculated using the MC method is itself a random variable, as different implementations of the MC method will produce different P_f values. The accuracy of the prediction will improve as the number of simulations increases. An estimate of the coefficient of variation for the calculated P_f can be determined using Eq. (33), in which N is the number of simulations in the MC method [60]. The calculated coefficient of variation for the prediction of P_f for the reference slabs both with and without shear reinforcement was $< 1\%$. This holds for all studied specimens. This indicates that the chosen number of interactions ($N = 10^6$) was sufficient for the study carried out in this article.

$$CoV_{P_f} = \sqrt{\frac{1 - P_f}{N \cdot P_f}} \quad (33)$$

Figure 11 shows the degree of correlation between the variability of the most significant input variables and the limit state variables, G_c and G_s . The correlation is characterised by Spearman's rank correlation coefficients [61] in the interval $[-1.0, 1.0]$. The Spearman's rank correlation coefficient measures the strength and direction of a monotonic association between two variables. The coefficient can detect non-linear relationships. A correlation coefficient close to 1 (or -1) indicates a strong positive (or negative) correlation. It was verified that all results presented in Figure 11 are statistically significant (p -value ≤ 0.05). Clearly, the uncertainty associated with the shear resistance models (θ_{VRc} and θ_{VRs}) had the greatest impact on the uncertainty distribution of the variables G_c and G_s . A greater value of θ_{VRc} (or θ_{VRs}) results in a greater value of G_c (or G_s), and thus lower probability of shear failure. The uncertainty in the model for bending capacity (θ_{MR}) was the second most significant factor. Consequently, properly quantifying the uncertainties related to the resistance models is essential to correctly estimating the probability of shear failure.

Besides the model uncertainties, the uncertainties related to the material strengths also had a noticeable influence on the resulting failure mode. The yield strength of the longitudinal reinforcing steel, f_y , is negatively correlated to the limit state variables. That is, a greater yield strength, which produces greater bending capacity, is likely to result in lower G_c and G_s , and thus higher probability of shear failure. In contrast, f_c is positively correlated to the limit state variables. That is, greater f_c results in lower probability of shear failure. This effect is significant in members without stirrups, whose shear capacity is proportional to the compressive strength.

While the bending capacity also increases with increasing f_c , this effect appears to be minor compared to the enhancement of shear capacity. However, the effect of f_c is marginal in members with shear reinforcement. Finally, the yield strength of the shear reinforcement, f_{yw} , is positively correlated to G_s . That is, a greater yield strength in the stirrups results in lower probability of shear failure.

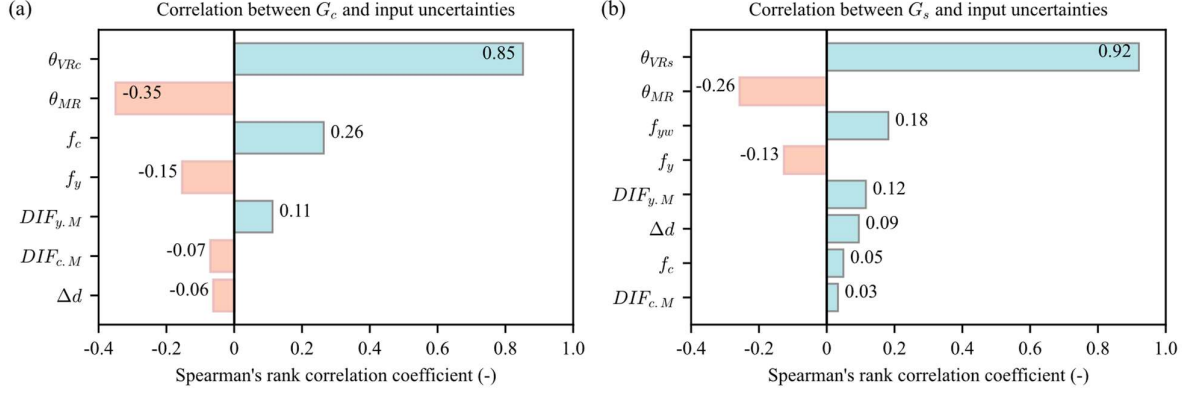


Figure 11. Correlation coefficients (Spearman's rank) between the uncertainty of input parameters and: (a) the limit state variable for the slab without shear reinforcement G_c , (b) the limit state variable for the slab with shear reinforcement G_s .

It is interesting to note that while the correlation between Δd and G_c in the slab without stirrups is negative, the correlation between Δd and G_s in the slab with stirrups is positive. The former is the combined influence of the size effect on the shear capacity and the variation of the longitudinal reinforcement ratio as a function of d . In both cases, greater d results in proportionally lower shear capacity. That is, while the shear capacity without stirrups does increase with d , it does so slower than linearly. In contrast, the positive correlation between Δd and G_s may be related to more stirrups being activated with greater d , which interacts with the effect of f_{yw} (which is significant and positive).

According to Figure 11, the variability of the DIFs appeared to have little effect on the outcome of the MC simulations. This variability stemmed mostly from the uncertainties associated with the chosen mathematical models for calculating the strain rates and dynamic increase factors. However, it should be noted that a significant bias may be introduced by the choice of DIFs used in the design of the RC element. This is discussed further in Section 5.3.

For the two reference specimens examined so far, which were selected as they have equal design resistance to shear and bending, the results show that introducing stirrups has the effect of decreasing the probability of shear failure. Results from numerical simulations reported by Kim et al. [62] support this outcome. Kim et al. [62] conducted simulations of RC beams subjected to high loading rate. In otherwise identical beams, the presence of stirrups was shown to stop the development of the main shear cracks. Furthermore, while the peak load was similar with and without stirrups, the post peak behaviour was clearly different. The beam without stirrup lost all carrying capacity when the main shear crack fully opened. Conversely, even though noticeable shear cracks developed in the beam with stirrups, they did not govern the overall response. Instead, the beam with stirrups could accommodate all the deformation necessary to absorb the imparted energy and did not reach failure during the simulations. These results indicate that in cases in which the shear resistance and flexural resistance are similar, the presence of stirrups can stop brittle failure and allow for a more ductile failure mode.

However, including shear reinforcement does not guarantee avoidance of shear failure. The experimental investigations of dynamic shear failure conducted by Somraj et al. [31] and Fujikake and Somraj [63] showed that in otherwise nominally identical RC beams, including a small amount of shear reinforcement increased the total resistance of the beam at different loading rates, but the beams still developed a shear failure, albeit less brittle than in the specimens without stirrups. However, further increase of the amount of shear reinforcement prevented shear failure altogether, allowing the beam to display a full flexural failure, characterised by a constant plastic capacity after reaching peak load. Even though the conditions in [31,63] are different from the beams studied here, these results indicate that adding stirrups has the potential of both delaying shear failure or preventing it altogether and thus enabling full flexural plastic capacity to develop.

5.2 Analysis of the entire sample

All specimens of the sample were subjected to the same analysis showcased in Section 5.1. Figure 12 shows the relationship between the probability of premature shear failure and the amount of longitudinal reinforcement. In general, members with shear reinforcement exhibited lower probability of shear failure. Indeed, only nine specimens with stirrups had $P_{fs} > 50\%$, while about half of the studied specimens without stirrups displayed $P_{fc} > 50\%$.

Clearly, the probability of shear failure increases with increasing reinforcement ratio, both in specimens with and without shear reinforcement. Indeed, the Sperman's correlation coefficient for these two parameters is equal to +0.93 and +0.90 for members with and without shear reinforcement (p -value ≤ 0.01). The main reason for the increased risk of shear failure is that a greater reinforcement amount results in greater flexural capacity and stiffer response, which in turn increases the shear demand of the element. This outcome agrees with experimental and numerical observations reported by Magnusson et al. [6] and Kamali [64]. The implication is that overly conservative designs with regard to flexural strength may increase the risk of premature shear failure. Furthermore, this suggests that stricter limits on the maximum reinforcement amount should be considered for blast-loaded beams. Indeed, some design regulations already impose lower values of the maximum allowed amount of longitudinal reinforcement in protective structures. For instance, the maximum allowed reinforcement amount according to the design regulation FKR 2011 for structures that require physical protection issued by the Swedish Fortifications Agency [65] is 0.50 %. Coincidentally, for all evaluated values of slenderness, bending failure (preferred) would be the dominant mechanism ($P_f < 0.5$) if ρ_l is less than around 0.45 %, which is close to the maximum limit imposed by FKR 2011. Another example is the Swedish design guidelines for civil defence shelters [66], which specifies a maximum reinforcement amount of 1.17 % for $f_{ck} = 35$ MPa and $f_{yk} = 500$ MPa. However, the maximum allowed reinforcement amount in other design regulations, such as Eurocode 2 and UFC 3-340-02, may be much larger.

Figure 12(a) also shows that in slabs without stirrups, the probability of shear failure decreased with increasing slenderness for a given reinforcement amount. For a constant d and reinforcement amount, slabs with longer spans (i.e. greater slenderness) are subjected to a decreased shear demand. However, besides slenderness, the effective height of the cross section also has an impact on the probability of shear failure, which is mainly related to the size effect on the shear resistance. That is, slabs with greater effective height will have a relatively lower shear resistance. This can be discerned in Figure 12(a) by comparing the results for different slenderness at $\rho_l = 0.50\%$. It can be seen that P_{fc} for slabs with $L/d = 11.9$ is closer to the P_{fc} corresponding to $L/d = 14.2$ than to P_{fc} for $L/d = 11.4$ and $L/d = 11.1$. This is mainly due to the slabs with slenderness $L/d = 11.9$ having $d = 252$ mm, compared to $d = 352$ and 452 mm for $L/d = 11.4$ and 11.1 , respectively.

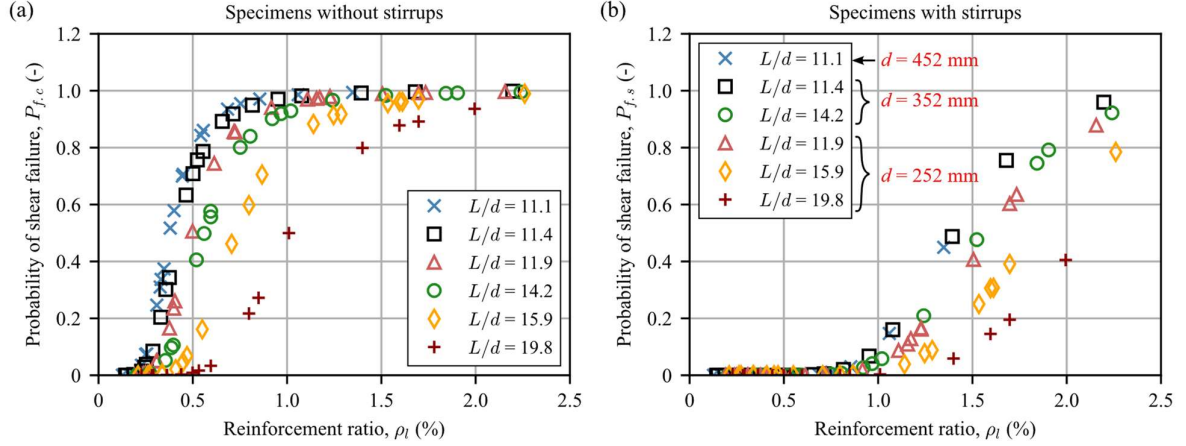


Figure 12. Probability of premature shear failure, P_f , as a function of the reinforcement ratio, ρ_l , and slenderness ratio (L/d).

The same trend regarding decreased probability of shear failure for increasing slenderness was also displayed by the slabs with stirrups. However, it appears that the effective height, d , also has an impact, considering that the values of $P_{f,s}$ for members with $L/d = 14.2$ ($d = 352$ mm) and $L/d = 11.9$ ($d = 252$ mm) are similar. The main reason for this is the limit imposed by Eurocode 2 on the minimum amount of shear reinforcement, which depends either on the material properties or on the effective height of the cross section, see Eq. (16). The minimum amount in specimen with $L/d = 14.2$ was given by the top alternative in Eq. (16), while the minimum amount in specimens with $L/d = 11.9$ was given by the bottom alternative in Eq. (16).

The influence of the applied impulse and the utilization of the deformation capacity is investigated in Figure 13. In general, visual inspection indicates that there is a trend for increased probability of shear failure as the imposed impulse increases. This is confirmed by the Spearman's rank correlation coefficient, which is +0.78 for members without stirrups and +0.67 for members with stirrups (p -value ≤ 0.01 for both cases). It is worth noting that while the *uncertainty* of the impulse does not seem to have a significant effect for a particular specimen (see Figure 11), the nominal value of the applied impulse does have a noticeable effect on P_f across the entire sample. Moreover, for a given impulse value, the probability of shear failure shows significant variability depending on the utilization of the deformation capacity. Slabs that are designed for low utilization of deformation capacity (which produces highly reinforced beams) exhibit greater probability of shear failure for a given impulse. In contrast, the probability of shear failure at a given impulse value decreases if the slabs are designed for high utilization of the deformation capacity (lightly reinforced beams). This negative correlation between probability of premature shear failure and utilization of deformation capacity is supported by the Spearman's rank correlation coefficient, which is -0.22 for members without stirrups (p -value = 0.02) and -0.19 for members with stirrups (p -value = 0.04). This implies that overly conservative design with regard to the deformation capacity of the slab results in increased risk for shear failure.

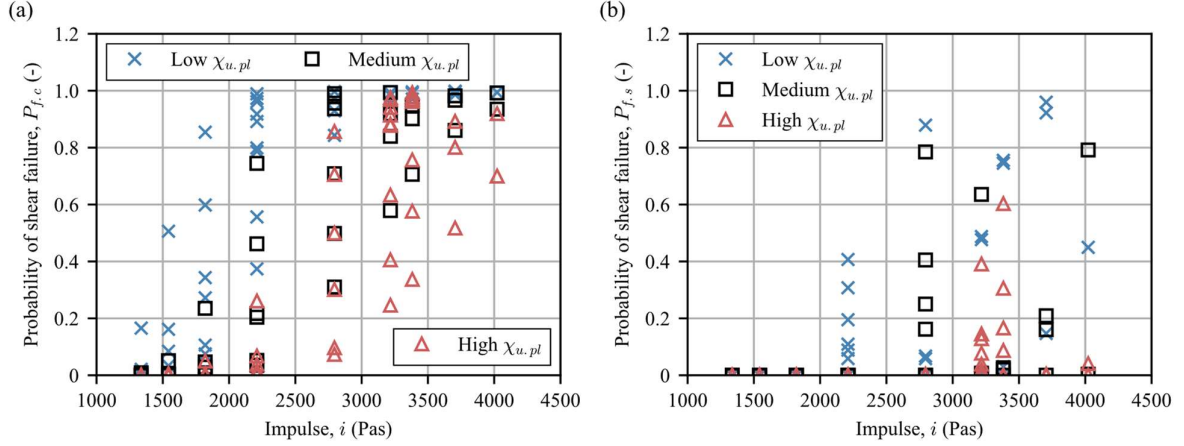


Figure 13. Probability of premature shear failure, P_f , as a function of the reflected impulse, i , and the utilization of plastic deformation capacity, $\chi_{u,pl}$. Low: $\chi_{u,pl} \leq 0.33$, Medium: $0.33 < \chi_{u,pl} \leq 0.67$, High: $\chi_{u,pl} > 0.67$.

The probability of shear failure of all specimens is plotted against the *design* utilization of shear capacity, χ_V , in Figure 14. The utilization of shear capacity is defined as the ratio between the design shear demand and the design shear capacity, according to Eq. (34). At a first glance, χ_V appears to be the inverse of the limit state variable G . However, it should be noted that, while G is a stochastic variable due to the uncertainties involved in the design process, χ_V is a deterministic parameter that depends on the choices made during design, including material factors and characteristics value of material properties. The references specimens studied in Section 5.1 had $\chi_V = 1.0$.

$$\chi_V = \frac{V_{E,d}}{V_{R,d}} = \frac{R_{M,d}}{R_{V,d}} \quad (34)$$

Figure 14 shows that the probability of premature shear failure of specimens designed to fulfilled $\chi_V \leq 1.0$ is $\leq 26\%$ if the element has no stirrups and $\leq 9\%$ if the element is provided with stirrups. This implies that the adopted design approach using the resistance models in Eurocode 2 has an intrinsic safety margin against shear failure in slender RC slabs. Indeed, shear failure first becomes the expected failure mode (probability of shear failure $> 50\%$) when the utilization ratio of shear capacity at design exceeds approximately 1.13 in members without stirrups and 1.42 in members with stirrups. Furthermore, while P_f is affected by the applied load, the geometry and the reinforcement amount, it appears that the relationship between P_f and χ_V is independent of those parameters.

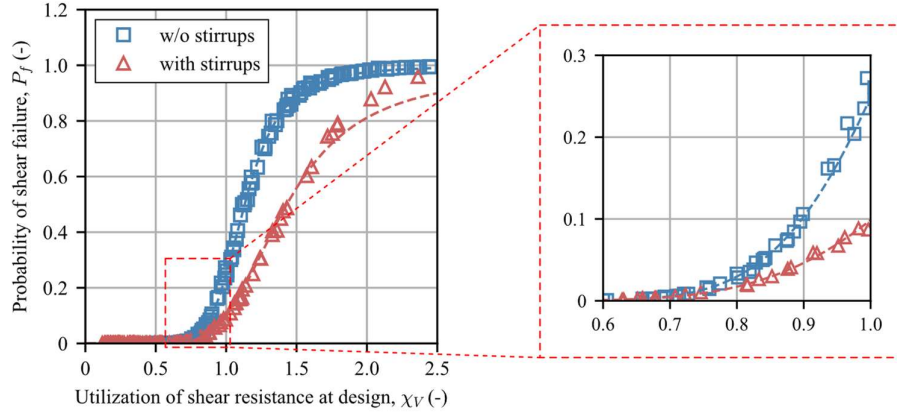


Figure 14. Probability of shear failure, P_f , as a function of the utilization of shear capacity at design, χ_V . The dashed lines represent the best fit of the logistic function given by Eq. (37).

5.3 Influence of the choice of DIFs during design

The results so far have shown that the variability related to DIFs appeared to have little effect on the limit state variable, see Figure 11. However, this variability refers to the uncertainty of the value of DIFs with respect to the nominal value, and thus it is a measure of the accuracy of the chosen mathematical models for calculating the strain rates and the dynamic increase factors. However, the initial choice of DIFs at design may introduce a significant bias on the outcome of the MC simulation. Essentially, if a different set of DIFs were used for the initial design of the slabs, a different value of the probability of shear failure would be obtained. This effect was further investigated for all slabs of type 1 ($L = 3000$ mm, $h = 300$ mm), see Table 1. Three approaches were considered at design: i) DIFs calculated based on the material property and strain rate (which is the main method used in this article); ii) DIFs taken from the design tables in UFC 3-340-02 ($\text{DIF}_{c,M} = 1.25$, $\text{DIF}_{y,M} = 1.23$, $\text{DIF}_{c,V} = 1.0$, $\text{DIF}_{y,V} = 1.1$), which are independent of material strength and strain rate; and iii) All DIFs set equal to 1.0, that is, strain rate effects are disregarded at the design stage. Regardless of the choice made during design, the dynamic strength was computed within the MC method according to the approach outlined in Section 2.3 (which is considered as the most accurate approach).

The results are presented in Figure 15. The specimens initially designed with the tabulated DIFs in UFC 3-340-02 display slightly higher probability of shear failure than the corresponding elements designed under the assumption of strain-rate dependant DIFs. Indeed, at $\chi_V = 1.0$, by using the DIFs from UFC 3-340-02 at design, the resulting probability of shear failure increased from $P_{f,c} = 26\%$ to $P_{f,c} = 31\%$ and from $P_{f,s} = 9\%$ to $P_{f,s} = 10\%$ in members without and with stirrups, respectively. Instead, if the DIFs were disregarded during design, the curve would be noticeably displaced towards the left side of the plot. That is, the probability of shear failure would be greatly increased. At $\chi_V = 1.0$, if all DIFs were set to 1.0 during the initial design, the probability of shear failure would be much higher: $P_{f,c} = 70\%$ and $P_{f,s} = 20\%$. Consequently, disregarding the strain rate effects during the initial design of the RC slabs may significantly enhance the risk of premature shear failure. The main reason for this is that the dynamic bending capacity of the structure is greater than the corresponding static capacity, and thus the equivalent shear demand at high loading rate also increases.

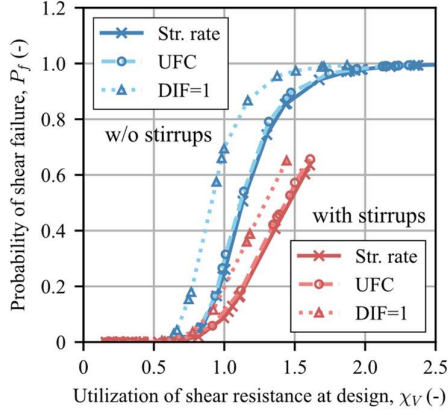


Figure 15. Effect of choice of DIFs at design on the probability of shear failure, P_f , as a function of the utilization of shear capacity at design, χ_V . The labels stand for: Str. rate = DIFs calculated based on the strain rate, UFC = DIFs taken directly from the design tables in UFC 3-340-02. DIF=1: All DIFs are equal to 1.0.

5.4 Reliability-based overstrength factor for shear design

Bending failure has been shown to be the most likely failure mode in the studied blast-loaded simply supported RC slabs. However, the risk of premature shear failure still cannot be ruled out with a sufficiently high confidence level. Indeed, the estimated probability of premature shear failure may be considered relatively high depending on the risk tolerability, particularly for beams with smaller slenderness ratio, higher longitudinal reinforcement amounts and no shear reinforcement.

To guarantee higher levels of confidence concerning the preferred type of failure, it is possible to introduce an overstrength factor, γ_{Rd} , in accordance with the principles of capacity design. Such overstrength factor is used to magnify the *design* shear demand, V_{Ed} , pertaining to the desired ductile response of the element. The shear capacity of a RC element subjected to blast loading should then fulfil the design condition:

$$V_{R,d} \geq \gamma_{R,d} \cdot V_{E,d}(M_{R,d}) \quad (35)$$

Furthermore, it is desirable to find a relationship between the value of the overstrength factor and a given reliability level. The relationship between the probability of shear failure and the utilization of shear capacity, χ_V , previously identified can be conventionally used to make an estimation of the overstrength factor. This is achieved by relating the overstrength factor to the utilization of shear capacity as

$$\gamma_{R,d} = \frac{1}{\chi_V} \quad (36)$$

The modified logistic function, defined by Eq. (37), was fitted to the data in Figure 14 using regression analysis. For specimens without shear reinforcement, $k = 9.54$ and $a = 0.88$. For specimens with shear reinforcement, $k = 7.34$ and $a = 0.70$.

$$P_f(\chi_V) = 1 - \frac{1}{1 + \exp[-k(\chi_V^{-1} - a)]} \quad (37)$$

Table 6 and Figure 16 give the calculated values of the overstrength factor for different reliability levels for blast-loaded RC slabs without stirrups ($\gamma_{Rd,c}$) and with stirrups ($\gamma_{Rd,s}$). The required overstrength factor is generally lower for members with shear reinforcement, though this difference diminishes as the reliability level becomes stricter. This factor is an important contribution of practical application for minimizing probability of shear failure in blast-loaded RC slabs designed according to the resistance models in Eurocode 2 and using reinforcement grade C. Moreover, the recommended values may be applied for RC members designed assuming the tabulated DIFs in UFC 3-340-02.

Table 6. Calculated overstrength factors to reduce the risk of premature shear failure for specimens without stirrups, $\gamma_{Rd,c}$, and with stirrups, $\gamma_{Rd,s}$.

Target probability of shear failure, P_f	$\gamma_{Rd,c}$ [-]	$\gamma_{Rd,s}$ [-]
$\leq 10.0 \%$	1.12	1.00
$\leq 5.0 \%$	1.20	1.08
$\leq 2.0 \%$	1.30	1.20
$\leq 1.0 \%$	1.37	1.28
$\leq 0.1 \%$	1.61	1.57

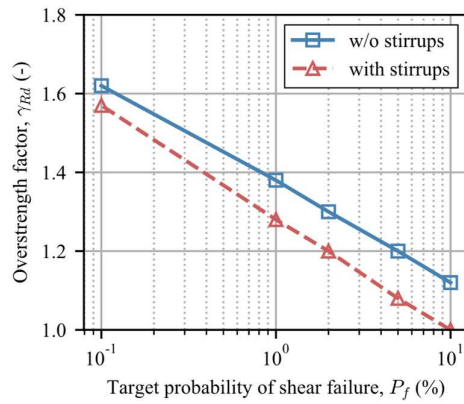


Figure 16. Variation of the calculated overstrength factor for different targets of probability of shear failure.

The choice of target reliability level depends on factors such as possible consequences of failure in terms of loss of life and economic losses and the costs required to reduce the risk of failure. Recommendations for target reliability levels for different design situations are given by such standards as the JCSS Probabilistic Model Code [45], Eurocode 0 [67] and ISO 2394 [68]. However, target reliability levels for the situation investigated here (i.e. shear failure should be prevented in favour of bending failure) are not clearly defined. Twisdale et al. [69] argue that practical and achievable probability of survival for most protective structures are of the order of 90 % to 99 % due to the major uncertainties inherent in the design. However, cost considerations may require designs that tolerate lower target reliability levels. In the context of this article, an analogy between the probability of survival ($1 - P_f$) and the probability of a bending mechanism dominating the failure of the RC member can be drawn. This means that suitable and achievable targets of the probability of shear failure occurring before bending failure may be of the order of 10 % to 1 %. Nevertheless, this article does not seek to recommend a given target probability of shear failure. Instead, the study presents values of the overstrength factor for different reliability levels to allow the user to use the value best suited to the conditions of their design.

The capacity design approach has been integrated in several seismic design regulations worldwide. According to this approach, the design action effects concerning brittle failures must be determined assuming that the

plastic mechanism with their possible overstrength has developed. It is of interest to compare the overstrength factors derived in this study to the overstrength adopted by different seismic codes for determining the design shear forces in primary seismic beams. Eurocode 8 [13] specifies a overstrength factor of 1.0 for structures with medium ductility class (DCM) and 1.2 for structures with high ductility class (DCH). The Chinese code GB50011-2010 [11] specifies overstrength factors of 1.3 for seismic class 1 (highest), 1.2 for class 2 and 1.1 for class 3. The New Zealand code NZS 3101 [12] specifies higher overstrength factors of 1.25 for reinforcement with seismic grade 300E and 1.35 for reinforcement with seismic grade 500E. Thus, the overstrength factors derived in this article are of the same order as the overstrength specified by different design regulations if the target probability of shear failure is 1% or greater. For stricter targets of probability of shear failure, the overstrength factor increases rapidly, mainly due to the long and flat tail of the uncertainty distribution of the limit state variable G .

Furthermore, adopting the overstrength factor for DCH structures in Eurocode 8 to reduce the risk of premature shear failure would lead to $P_{fc} \leq 5\%$ in RC slabs without shear reinforcement and $P_{fs} \leq 2\%$ in RC slabs with shear reinforcement.

6 Conclusions

Reinforced concrete (RC) structures exposed to blast loading are primarily designed to dissipate the imposed energy by means of large deformations rather than relying on a large load-carrying capacity. Consequently, a premature brittle shear failure must not limit the energy absorption capacity that a ductile bending failure can provide. However, ensuring that a shear mechanism does not occur is difficult due to the uncertainties related to the material and geometrical properties, and the different models for calculation of resistance values, blast load parameters and strain rate effects. This study implemented the Monte Carlo (MC) to determine the probability of premature shear failure (before bending failure) in simply supported slender RC one-way slabs subjected to blast loading. The RC slabs were designed using the resistance models for statically loaded structures in Eurocode 2, in combination with dynamic increase factors (DIF) to account for strain-rate effects and a single-degree-of-freedom (SDOF) model to predict the dynamic response. The main conclusions from the study are as follows:

- The probability of premature shear failure in beams designed according to the resistance models in Eurocode 2 was found to be less than or equal to 26 % in members without shear reinforcement and less than or equal to 9 % in slabs with shear reinforcement. That is, the studied slabs are more likely to fail under bending, which suggests that there is an intrinsic safety margin concerning the preferred failure mode in the adopted design procedure. Furthermore, the confidence level regarding the preferred failure mode is higher for members with shear reinforcement.
- The probability of premature shear failure was shown to be strongly positively correlated to the amount of longitudinal reinforcement and applied impulse. That is, specimens with greater reinforcement amount or subjected to greater impulse are at greater risk for premature shear failure. This was confirmed by the Spearman's rank correlation coefficient, which is greater than +0.90 for the former and greater than +0.67 for the latter. This implies that stricter upper limits on the allowable amount of longitudinal reinforcement should be considered for blast-loaded RC elements.
- Disregarding or significantly underestimating the influence of strain rate effects on the material strength under bending action at the design stage may considerably increase the risk for premature shear failure of blast-loaded RC slabs.
- Among all considered sources of uncertainties, the uncertainties related to the resistance models in Eurocode 2 were found to have the most considerable influence on the probability of premature shear failure of blast-loaded RC slabs. This emphasises the need to conduct more exhaustive studies of model uncertainties for this type of dynamic loading.
- Even though bending failure was found to be the most likely failure mode, the probability of developing a shear failure with limited deformation capacity (and thus low energy absorption capacity) may still be too high, particularly for members without shear reinforcement. A safer design of RC one-way slabs subjected to blast loading regarding premature shear failure may be achieved by magnifying the design shear force with an overstrength factor. This article presented values of the overstrength factor as a function of the desired reliability level for obtaining a ductile failure. In members without stirrups, the factor varies between 1.12 and 1.61 for achieving a probability of premature shear failure less than 10% and less than 1%, respectively. In members with stirrups, the overstrength factor varies between 1.0 and 1.57.

This work was based on simplified analytical models for estimating both the resistance and dynamic response of the RC slabs. Future research work using more advanced methods, such as non-linear finite element analysis (FEA), are required to confirm the results. It is of particular interest to use advanced non-linear FEA to gain

better understanding regarding the dynamic shear resistance of slender RC slabs and for verifying the suitability of the shear resistance models in Eurocode 2 for high loading rate conditions.

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