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Estimating safe scaled distances for columns subjected to blast

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This paper is concerned with determining the minimum stand-off distance required to prevent column failures in reinforced concrete framed buildings. This is of interest because column failures can initiate progressive collapse, resulting in mass casualties. A technique is developed to determine the critical range at which failure will occur for a given weight of explosives and thus provide a safe scaled distance. The method is used to carry out a parametric study of a range of reinforced concrete columns of variable dimensions and strengths. The corresponding data were used to predict safe scaled distances for columns (with and without clearing). These values can be used to estimate the minimum stand-off distance required to prevent progressive collapses of buildings that may be subjected to deliberate blast loading.

Notation

A_g	cross-sectional area of column
B	column breadth
D	column depth
EI	flexural stiffness
f_{ck}	characteristic cube compressive strength
I_g	second moment of area for gross concrete section
I_s	second moment of area of the transformed area of steel reinforcement
I_t	second moment of area for the transformed section
K_s	normalised shear strength
L	column length
m	mass per unit length
n	mode number
$P(t)$	blast load on column per metre length at time t
q	modal amplitude
SD	stand-off distance
u	displacement
V_u	column shear strength
W	charge weight
ϕ	mode shape
ω_n	natural frequency of mode n

1. Introduction

The most effective means of protecting buildings against vehicle-borne improvised explosive devices (VBIEDs) is to maximise the distance between the explosive and the structure. This is often ensured by the installation of barriers, such as the soil-filled (HESCO) gabions used by NATO expeditionary forces. The distance between the barrier and the building is commonly termed 'set-back' in the USA and 'stand-off' in the UK. The Hopkinson and Cranz scaling laws (Cranz, 1926; Hopkinson, 1915) show that over-pressure can be predicted from the range (R) divided by the cubed root of the mass of the explosive ($W^{1/3}$). This leads to the concept of scaled distance, $Z = R/W^{1/3}$, in which the over-pressure produced by different charge sizes would be identical for the same scaled distance (Z). Since damage is a function of over-pressure, the scaled distance is a useful parameter for setting stand-off distances because it allows the mass of the explosive to be varied.

Guidance as to what value stand-off should take varies, with the US Department of Defense (US DoD, 2004) recommending a scaled distance of not less than 4·46 m/kg$^{1/3}$ (11·24 ft/lb$^{1/3}$). This can be a restrictive limit and would, for example, require 45 m of stand-off to be provided for a design charge weight of 1000 kg (TNT). Within this range, masonry load-bearing buildings are at significant risk of collapse, although reinforced concrete (RC) framed buildings are known to survive at scaled distances well within this limit. UK research into blast effects on steel and RC framed buildings during World War II was led by Sir Dermot Christopherson, who suggested 'the region $1 < Z < 5$ ft/lb$^{1/3}$ [$0{\cdot}4 < Z < 1{\cdot}98$ m/kg$^{1/3}$] is of first-rate importance to the designer' (Christopherson, 1945). At a range of less than 0·4 m/kg$^{1/3}$, the target was defined as being within the explosive flame and outside the scope of normal design measures, whereas the upper limit of 1·98 m/kg$^{1/3}$ is of interest because, within this limit, failure of framing members is likely.

Buildings subjected to large blasts include the Marriott Hotel in Islamabad, which was subjected to a 4·4 m/kg$^{1/3}$ (11 ft/lb$^{1/3}$) blast in 2008, and the Police and Internal Affairs Building in the Republic of Ingushetia, Russia, which received a 2·4 m/kg$^{1/3}$ (5·3 ft/lb$^{1/3}$) blast in 2009. Neither of these buildings suffered structural damage to their supporting frames due to blast. Many progressive collapses of framed buildings subjected to VBIEDs have been triggered by the failure of columns, as occurred in the Alfred P. Murrah Federal Building in 1995, in which the failure of three perimeter columns led to the collapse of nearly half of the floor area. The scale of this disaster has previously been blamed on the use of a transfer girder to support every other perimeter column (Corley *et al.*, 1998). However, recent blast analysis of the building, in which the transfer girder was removed to allow all perimeter columns to continue down to ground floor level, has demonstrated that the VBIED detonated would still have created a 42 m wide collapsed section of the building (Byfield and Paramasivam, 2011). This work demonstrated that lack of redundancy was key to the collapse, because the building used open-plan architecture with a fully glazed façade.

Knowledge of the scaled distance at which RC columns begin to fail is of practical interest in the recommending of a bare minimum stand-off distance for use in congested urban areas. This paper aims to determine this stand-off distance by exploring the failure of RC columns. A method is presented for determining at what scaled distance the balance point between survival and failure occurs – this is termed the safe scaled distance (SSD).

2. The SSD approach

In the present work, estimation of the SSD for a given column and threat is by iteration. For a given charge weight, an initial range is assumed and blast wave parameters (e.g. reflected pressure) are determined. If the dynamic analysis reveals that the shear force is greater than the shear strength or the moment is greater than the flexural strength, then the column is deemed unsafe and the stand-off distance increased. This procedure is repeated until the strength of the column is exactly equal to the dynamic force developed in the column. This process is explained graphically in Figure 1 for the case of shear failure with a constant charge weight. The dynamic shear force experienced by the column and the shear capacity are non-dimensionalised by dividing by the total applied load, which is equal to the column face area multiplied by the peak reflected pressure. These normalised forces are plotted against the ratio between the blast time duration and the natural period of vibration of the column (t_d/t_n). This ratio governs the degree of impulsiveness and it also affects the ratio between shear strength and load, since the loading falls as the range increases.

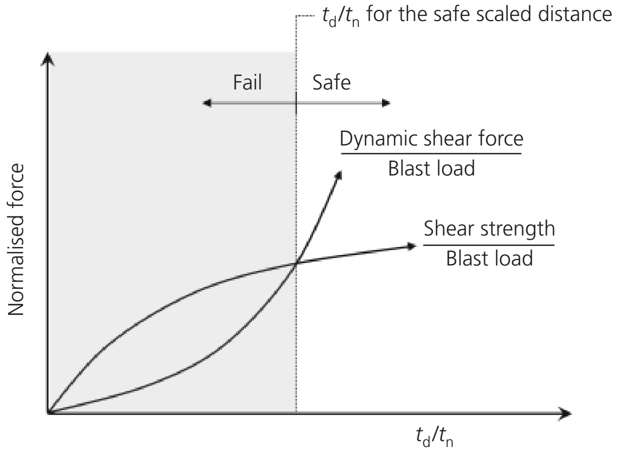

Figure 1. Safe stand-off distance considering shear failure

The intersection between the two graphs provides the value of t_d/t_n, which corresponds to the SSD. Once the critical t_d/t_n ratio is defined, it is then a trivial procedure to define the range that corresponds to the given charge weight of explosives.

3. Calculating the dynamic shear force and moment

During the assessment of shear failure, the columns were assumed to remain linear-elastic up until shear failure at the supports. This assumed response is supported by observations from bomb-damaged structures. The forensic investigation of the Murrah Building showed that two perimeter columns failed by shear and the closest to the blast failed by brisance. The perimeter columns that failed by shear were found to have remained largely elastic along their lengths (Corley *et al.*, 1998). Similar failures were observed during forensic investigations carried out on bomb-damaged buildings during World War II (Byfield, 2004, 2006; Smith *et al.*, 2010). Figure 2 shows bomb-damaged columns that experienced shear failures at ceiling level but which remained uncracked along their lengths. The assumption is conservative because a linear-elastic response produces a shorter natural period of vibration than an elastic-plastic response. This leads to a higher dynamic shear force because the ratio of t_d/t_n is increased, which means the loading is less impulsive. In other words, by remaining elastic, shear failure at the supports is able to occur faster and at a lower load. In contrast, the dynamic analysis for flexural response was carried out assuming an elasto-plastic response.

The analysis reported here demonstrated that shear failure of columns always occurred at lower scaled distances than for flexural failure. Since shear failure was always found to be critical, detailed information on the analysis for flexure is not presented in this paper, but is available elsewhere (Paramasivam, 2008).

The columns were modelled using the standard distributed mass method defined in many textbooks (e.g. Biggs, 1964; Chopra, 2001). In this method, a partial differential equation (Equation 1) is developed using force equilibrium and the response $u(x, t)$ is

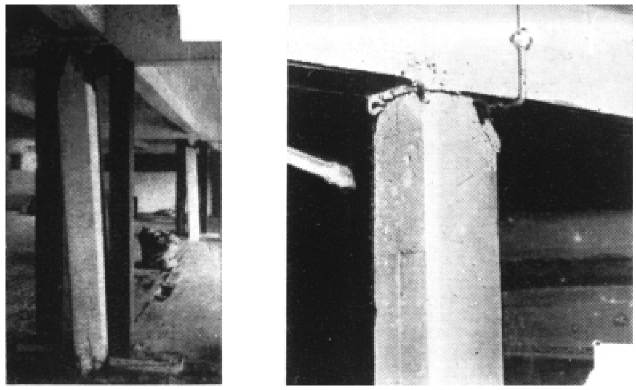

Figure 2. Shear failures due to blast (Baker *et al.*, 1948)

expressed as the superposition of the response of individual modes. For RC columns, the shear deformation and the effect of axial compression on the dynamic response are negligible and hence ignored.

$$m\frac{\partial^2 u}{\partial t^2} + \mathrm{EI}\frac{\partial^4 u}{\partial x^4} = P(x,\, t) \tag{1}$$

$$u(x,\, t) = \sum_{n=1}^{\alpha} \phi_n(x)_n q(t) \tag{2}$$

where ϕ represents the mode shape and q represents the modal amplitude.

In this study, the response u was obtained by combining these standard time and spatial functions to form the following expressions for columns

$$\begin{aligned} u(x,\, t) = {} & \frac{-4PL^3}{\pi^5 \mathrm{EI}} \\ & \times \sum_{n=1,3,5,\ldots}^{\alpha} \frac{1}{n^5}\left(\frac{\sin(\omega_n t)}{\omega_n t_\mathrm{d}} - \cos(\omega_n t) + \frac{t_\mathrm{d} - t}{t_\mathrm{d}}\right) \\ & \times \sin\left(\frac{n\pi x}{L}\right) \end{aligned} \tag{3}$$

$$\begin{aligned} u(x,\, t) = {} & \sum_{n=1,3,5,\ldots}^{\alpha} \left(\frac{u_n(t_\mathrm{d})}{\omega_n} \sin[\omega_n(t - t_\mathrm{d})] \right. \\ & \left. + \, u_n(t_\mathrm{d}) \cos[\omega_n(t - t_\mathrm{d})]\right) \\ & \times \sin\left(\frac{n\pi x}{L}\right) \end{aligned} \tag{4}$$

where

$$\begin{aligned} u_n(t_\mathrm{d}) = {} & \frac{2P}{mL\omega_n^2 \pi} \frac{[(-1)^n - 1]}{n} \\ & \times \left(\frac{\sin(\omega_n t_\mathrm{d})}{\omega_n t_\mathrm{d}} - \cos(\omega_n t_\mathrm{d})\right) \end{aligned} \tag{5}$$

$$\begin{aligned} u_n(t_\mathrm{d}) = {} & \frac{2P}{mL\omega_n \pi} \frac{[(-1)^n - 1]}{n} \\ & \times \left(\frac{\cos(\omega_n t_\mathrm{d})}{\omega_n t_\mathrm{d}} - \sin(\omega_n t_\mathrm{d}) - \frac{1}{\omega_n t_\mathrm{d}}\right) \end{aligned} \tag{6}$$

in which L is the length of the column, m is mass per unit length, n is the mode number, $P(t)$ is the blast load on the column per metre length at time t, u is displacement, ω_n is the natural frequency of the nth mode and EI is flexural stiffness.

If the duration of the blast is quasi-static, the maximum response occurs before the load diminishes to zero; the maximum response of an idealised column pinned at both ends was calculated using Equation 3. This equation was used up to time t_d. If the maximum deflection occurred after the load diminished to below zero, the response occurring after time t_d was calculated using Equation 4. Shear forces and moments were calculated from these displacement functions by solving the governing beam equations.

During the analysis for shear, the uncracked transformed section moment of inertia was used in the calculation of the natural time period ($I_\mathrm{g} + I_\mathrm{t}$), I_g being the gross moment of inertia and I_t the second moment of the transformed area of reinforcement. During the analysis for flexural failure, the cracking of concrete is important in reducing stiffness and increasing the natural time period of vibration and thus reducing peak moments on the column. In the present analysis the US DoD approach (US DoD, 2008) for flexure of using the average moment of inertia of the cracked and uncracked sections was used.

4. Defining blast loading

This investigation considered conditions in which the reflected pressure was or was not free to clear either side of the column. In columns that are free to clear, the net load on the column (see the shaded area in Figure 3(c)) is the front-side load (Figure 3(a)) minus the rear-side load (Figure 3(b)). Various graphs to estimate air blast loads are available (Brode, 1955; Henrych, 1979; Kingery and Bulmash, 1984). In the present investigation, blast wave parameters were based on the equations developed by Kingery and Bulmash (1984). Load duration is governed by the velocity of the wave front and the clearance distance, as the blast wave clears quickly if the blast wave is free to wrap around the column. For the work presented in this paper, the complex pressure–time history shown by the shaded region in Figure 3(c) was converted into a simple triangular pulse of equivalent area (impulse), as sketched in Figure 3(d). The time duration (t_d) for the triangular pulse was calculated by dividing the impulse (net area under pressure–time curve) by the peak reflected pressure. In other words, t_d is the effective time duration inclusive of clearing effects. To study the inaccuracy resulting from this simplification, a parametric study was carried out (Paramasivam, 2008), which showed that the approximation is conservative and the error is within 10% of the response. The reflected wave from the adjacent beams and slabs either reduces the blast load on the column or hits the column after the direct load diminishes. Conservatively, this effect is ignored.

5. Application of the method to a real building

The Murrah Building incident was reanalysed to illustrate the approach. The blast load W and its duration t_d were estimated for

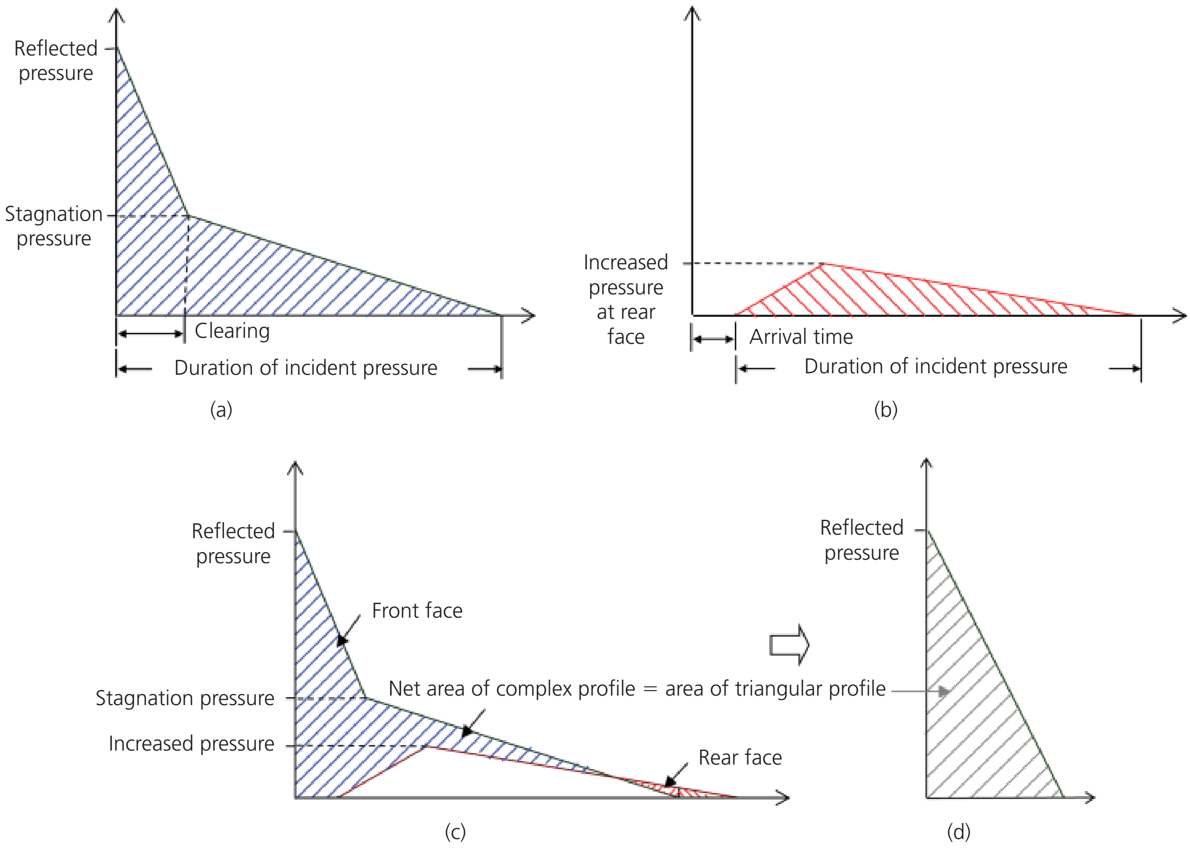

Figure 3. (a) Blast pressure on front face. (b) Blast pressure on rear face. (c) Net pressure on column. (d) Idealised blast load on column

various stand-off distances (3–50 m) with the charge size corresponding to that used in the original incident (1800 kg TNT equivalent). The shear capacity of the columns was calculated assuming an axial compression force resulting from full unfactored dead loads plus a third unfactored imposed load (BSI, 2000). Using the approach shown in Figure 1, the normalised shear force and strength were plotted against t_d/t_n. The intersecting point shown in Figure 4(a) occurred at $t_d/t_n = 0{\cdot}17$, which yielded a safe stand-off distance of 23·75 m (Figure 4(b)). The forensic investigation of the building showed that column G16 (range 15·24 m, scaled distance 1·25 m/kg$^{1/3}$) failed in shear, whereas the next closest column G12 (range 27·13 m, scaled distance 2·23 m/kg$^{1/3}$) survived. This corresponds to what was observed on the actual building and provides some confidence in the solution. Shear failure was found to be critical over flexural failure and the SSD for the building (against progressive collapse) was shown to be 1·95 m/kg$^{1/3}$, which is remarkably close to Christopherson's value of 1·98 m/kg$^{1/3}$.

6. Parametric study to determine a general design value for the scaled distance for RC columns

This method for calculating the SSD is complex and not for rapid use. It therefore becomes necessary to create interpolation functions for predicting the SSD for a given set of input parameters. The first step in developing an expression is to identify the influencing parameters. The dynamic shear force can be mathematically represented as follows. For clearing

7. $$V = f_1\left(W, \frac{B}{L}, \frac{D}{L}, L, \frac{I_t}{I_g}, f_{ck}, \mathrm{SD}\right)$$

and for non-clearing

8. $$V = f_2\left(W, \frac{D}{L}, L, \frac{I_t}{I_g}, f_{ck}, \mathrm{SD}\right)$$

where I_t is the second moment of area for the transformed section, I_g is the second moment of area for gross concrete section, B is the column breadth, D is the column depth, L is the column length, f_{ck} is the characteristic cube compressive strength, SD is the stand-off distance and W is the charge weight

For simplicity, this is not broken down into subparameters such as area of compression steel, cover thickness, etc. The explanation

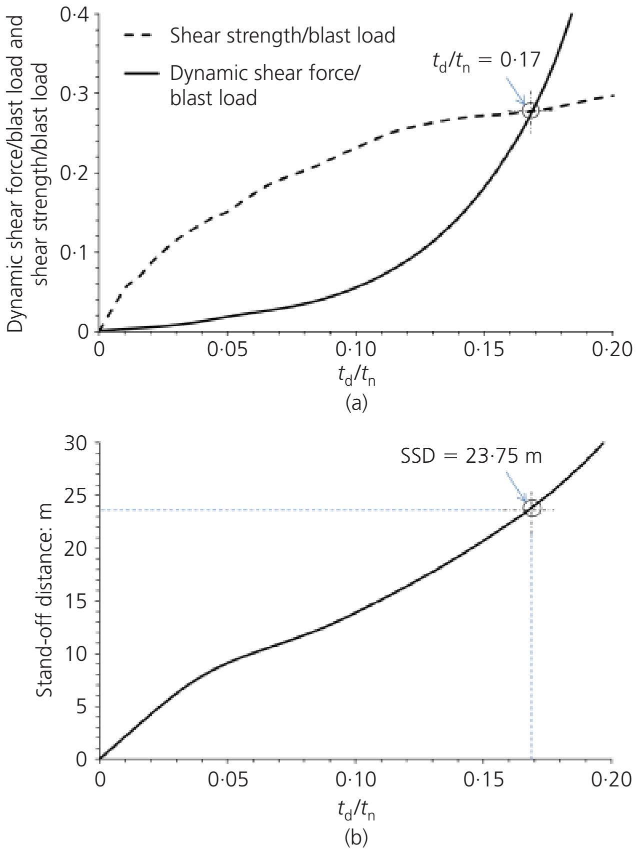

Figure 4. (a) Calculation of critical t_d/t_n ratio for the Murrah Building. (b) Calculation of SSD from the given t_d/t_n ratio

for the decision to divide B by L and D by L in this list of influencing parameters was that it facilitates reducing wide ranges of column sizes to the non-dimensional parameters from 0·05 to 0·30. This has numerical advantages in speed of calculation. The column shear strength V_u is normalised by dividing by $BD(f_{ck})^{1/2}$ to form the normalised shear strength K_s represented by Equation 10. This normalisation is advantageous because it provides a measure of the relative shear strength of the column, independent of column dimensions

9. $$K_s = \frac{V_u}{BD(f_{ck})^{1/2}}$$

therefore

10. $$V_u = K_s(f_{ck})^{1/2}\left(\frac{B}{L}\right)\left(\frac{D}{L}\right)L^2$$

This can be represented as

11. $$V_u = f_2\left(\frac{B}{L}, \frac{D}{L}, L, f_{ck}, K_s\right)$$

Since the dynamic shear force (V) ⩽ shear capacity (V_u)

$$f_1\left(W, \frac{B}{L}, \frac{D}{L}, L, \frac{I_t}{I_g}, f_{ck}, \mathrm{SD}\right) \leqslant f_2\left(\frac{B}{L}, \frac{D}{L}, L, f_{ck}, K_s\right)$$

To solve this equation, f_1 and f_2 are plotted against stand-off distance, hence W, B/L, D/L, L, I_t, f_{ck} and K_s are constant, and the intersection point (Figure 1) can be obtained. The SSD is mathematically represented by Equation 12 for clearing and Equation 13 for no clearing

12. $$\mathrm{SSD_{shear}} = f\left(W, \frac{B}{L}, \frac{D}{L}, L, \frac{I_t}{I_g}, f_{ck}, K_s\right)$$

13. $$\mathrm{SSD_{shear}} = f\left(W, \frac{D}{L}, L, I_t, f_{ck}, K_s\right)$$

From these equations it is clear that the SSD as related to shear failure depends on charge weight, concrete strength, B/L, D/L, L, I_t and K_s.

Using the above method, a database of SSDs for shear and flexure was generated using the following range of variables

- charge weight {230, 500, 1000, 1500, 1800, 4500 kg}
- f_{ck} ranging from 20 to 40 N/mm^2 (the concrete strengths are low, particularly for old structures in developed countries; however, the 20–40 N/mm^2 range is more realistic for buildings in developing regions where concrete strengths are often lower)
- B/L ranging from 0·05 to 0·3
- D/L ranging from 0·05 to 0·3
- L ranging from 2·5 to 6 m
- normalised shear strength K_s ranging from 0·1 to 0·8
- normalised flexural strength ranging from 0·1 to 0·8.

This provided an array of approximately 300 000 data points on which a regression analysis was based. A series of regression analysis functions was developed and used in a parametric study to explore the range of SSDs for columns adopted in practice. The functions (SSD equations) yielded maximum R^2 values of 0·996 and 0·994 to estimate the SSD of a column for clearing and no clearing, respectively. A total of 396 columns was examined based on a combination of the following parameters

- *B* {300, 450, 600 mm}
- *D* {300, 450, 600 mm}
- *L* {3, 4, 5, 6 m}
- diameter of main reinforcement {20, 25, 32 mm}
- number of main rebars {4, 8}
- two and four-legged ties with 12 mm rebars, spacing of ties {150, 200, 250 mm}
- axial compression of 10%, 20% and 30% of $A_g f_{ck}$.

The shear and flexural strength of these columns was estimated using BS 8110 (BSI, 1997) without any material safety factors. Normally, blasts produce load associated with strain rates in the range of 10^2–10^4 s^{-1} (Ngo *et al.*, 2007). This high strain rate increases the static compressive strength of the concrete. The US DoD (2008) introduced dynamic increase factors (DIFs) for steel and concrete to account for the effect of high strain rate. The DIF is defined as the ratio of dynamic to static strengths and it increases with strain rate. Recognising the difficulty in estimating strain rate, the US DoD (2008) proposed a conservative DIF for design purposes (1·25 for concrete and 1·10 for steel). In this paper, the strengths of concrete and steel were kept as variables. A designer can incorporate this effect by increasing the strengths of concrete and steel by the DoD factors.

7. Results from the parametric study

Figure 5 shows the SSD plotted against the normalised shear strength of the 396 columns considered, each with and without clearing. An equivalent graph is available in the literature showing SSD plotted against the normalised flexural strength (Paramasivam, 2008). Shear failure tends to be critical for columns due to a number of factors (for example columns tend to have low shear resistance and the higher modes of vibration contribute significantly to the dynamic shear force), whereas the higher modes of vibration do not contribute significantly to the dynamic bending moments. Hence the flexural graph is not included here.

The mean value of SSDs determined from these data was 1·84 m/kg$^{1/3}$ with clearing and 2·20 m/kg$^{1/3}$ without clearing. It is interesting to compare these values with those from other sources. As discussed previously, Christopherson (1945) concluded from his extensive investigations that the scaled distance of most interest to designers was within the limit of 1·98 m/kg$^{1/3}$. More recently, finite-element modelling using LS-Dyna by Wu and Hao (2007) provided a critical SSD of 1·8 m/kg$^{1/3}$ when column axial forces were low and 1·18 m/kg$^{1/3}$ when column axial forces were high. The reason for the variation with axial compression is because compression enhances the shear strength of the column. This parametric study assumed that axial compression was developed from a loading of 1·0 × dead load plus one third of the imposed load, and this produced axial compression forces of the order of 10–25% $A_g f_{ck}$. This corresponds to a low value of axial force from the Wu and Hao (2007) study; hence the scaled distance of 1·8 m/kg$^{1/3}$ is the most appropriate value for comparison purposes. Therefore, the results of both Christopherson and Wu and Hao are close to those found in the present investigation. The forensic analysis presented in this study indicated that the Murrah Building required an SSD of 1·95 m/kg$^{1/3}$, which again fits comfortably with the values of Wu and Hao (1·8 m/kg$^{1/3}$) and Christopherson (1·98 m/kg$^{1/3}$).

The US DoD (2004) provide an SSD value of 4·46 m/kg$^{1/3}$ for unstrengthened framed structures. This appears to be conservative, at least in respect of the column failure criteria. However, this value is designed to provide a basic level of protection from effects such as secondary fragmentation, which are not within the scope of this parametric study. In the congested urban environment, achieving an SSD of 4·46 m/kg$^{1/3}$ can be problematical, in which case a basic level of protection against column failure is of interest. In this circumstance, the SSD can be established approximately for a given column using the normalised shear strength plotted against SSD design chart presented in Figure 6. This shows the trend lines obtained from the parametric values presented in Figure 5. It should be noted that uplift can occur during blast loading and this will reduce the compression force in the columns. Therefore, the reduced shear strength due to loss of

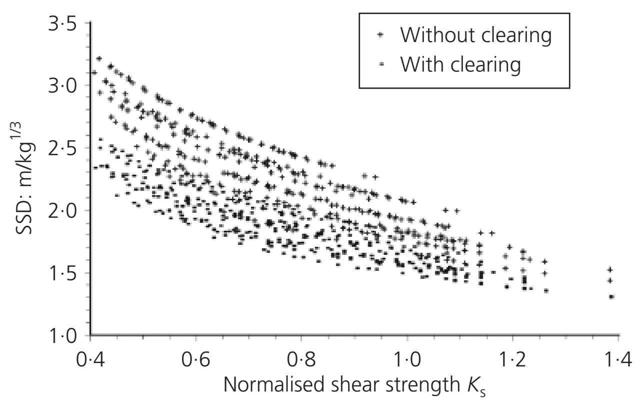

Figure 5. Normalised shear strength plotted against SSD

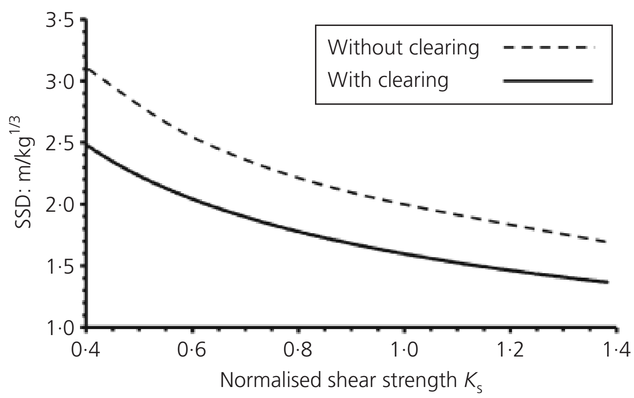

Figure 6. Normalised shear strength plotted against SSD design chart for RC columns

compression may need to be included when calculating the normalised shear strength for use with this chart. It should also be noted that this chart contains no factors of safety.

8. Conclusions

The SSD within which RC columns can be expected to fail is of interest when developing security measures for buildings because column failure is normally required before progressive collapse can be initiated. This paper has developed an iterative method for establishing the critical distance within which RC columns can be expected to fail. This distance is presented in terms of scaled distance and the approach has the built-in flexibility to account for variations in charge size. Using this method, a series of functions was developed from which the SSD could be determined for a given charge size, column geometry and set of material properties. These functions were used to establish the SSDs for a set of 396 columns, each with and without clearing of blast pressure. The results were used to develop a design chart for estimating the SSD for columns. This allows for a rapid estimation of the stand-off distance required to safeguard against column failure, inclusive of an allowance for the relative strength of the column. The results from the study have been found to be in good agreement with other studies, which included a World War II investigation into the performance of structures subjected to blast and a more recent study based on LS-Dyna. The method was also shown to predict correctly the failure modes that occurred when the Murrah Building was subjected to blast loading.

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