**Numerical modelling of reinforced concrete columns subject to coupled uplift and shear forces induced by internal explosions**

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**Abstract**

This paper investigates the effect of time-variant coupled uplift forces and lateral blast pressures on the vulnerability of reinforced concrete columns when subjected to internal explosions. Detailed examination of archived events have demonstrated that confined blast can induce significant uplift forces on concrete columns carrying predominantly compression loads. These uplift forces can deteriorate the strength of concrete columns leading to destabilization and a critical loss of structural adequacy. This research employs the Applied Element Method together with high resolution CFD simulations to model column response due to complex vented and contained internal blast environments. Results from a number of parametric studies highlighted the important effect of uplift forces and blast wave confinement with regard to the column’s overall vulnerability to internal building detonations. This research will be of direct importance to both practitioners and researchers involved with protective design of buildings.

**Keywords:** internal explosion, uplift force, applied element method, non-linear dynamic response, blasts

1. **Introduction**

Currently, there exists a great concern in the research community towards the destabilizing effect of tensile uplift forces on reinforced concrete (r.c.) columns due to blast wave confinement following internal building detonations. Despite the fact that time-dependant uplift forces induced by internal explosions act as a strength degrading mechanism for concrete columns ([Turgut, Gurel, & Pekgokgoz, 2013](#_ENREF_28)), to date there are no comprehensive studies dedicated to rigorously understand this phenomenon. As a consequence, the behaviour of concrete columns due to coupled uplift and lateral blast loads following internal building detonations remains poorly understood and a source for unconservative design assumptions. It is further acknowledged that the presence of strong boundary walls in buildings substantially intensify the magnitude of blast effects due to repetitive reflections ([Needham, 2010](#_ENREF_17)). The consequence of these blast wave magnifications on the response and survivability of internal concrete columns continues to be of topical importance to both practitioners and researchers.

A study of the current codified design procedures ([British Standards Institution [BSI], 1997](#_ENREF_8); [CEN, 2002](#_ENREF_9)) and guidelines for designing blast resistant structures ([US Army, 2008](#_ENREF_29)) reveal that the presence of simultaneous upward tension forces reduce column shear capacity in all design cases (quasi-static, dynamic and impulsive). This complex blast-structure interaction requires accurate modelling to fully understand structural response and thus, behaviour of building structures when concrete columns in internal blast environments are assessed. The estimation of blast loads and subsequent column response following internal building detonations is a difficult task at the limits of computational complexity. Simple load calculation methods ([Hyde, 1992](#_ENREF_11); [US Army, 2008](#_ENREF_29)) and response prediction methods such as the single-degree-of-freedom (sdof) technique ([Biggs, 1964](#_ENREF_6); [US Army, 2008](#_ENREF_29)) can address this problem in a simplistic and conservative manner. However, these techniques remain inadequate towards capturing the accurate behaviour of reinforced concrete members, associated with material fracture and fragmentation due to high strain rate loading conditions. Although blast trials are of significant interest among many researchers, such experimental trials require limited access and specialist facilities; in many cases costing prohibitive monetary sums. These challenges have led to a dilemma regarding what numerical methods should be utilized for modelling column response due to internal blast loads; either (i) simple chart-based estimation methods coupled with sdof or (ii) alternative approximate methods for modelling column response or, (iii) more complex high fidelity techniques demanding substantial time and effort in order to limit inadequacies in final design procedures.

In-depth investigations towards the response of reinforced concrete structures due to blast loading conditions require to be based on advanced numerical simulations including 3-D Computational Fluid Dynamics (CFD) or hydrocodes to estimate blast loads and sophisticated numerical models to precisely predict structural response. Such computational methods can be categorized into two broad groups: coupled and uncoupled methods. Coupled analyses consider pressure release and redistribution following localized failure or large deformations of structural members, and they in turn provide the most accurate representations of structural response. However, such advanced methods are not always necessary because neglecting large deformation-induced pressure redistributions lead to conservative load cases, and the uncertainties that exist in the determination of design blast loads do not warrant such accuracy ([Williamson et al., 2010](#_ENREF_32)). In a comprehensive numerical investigation conducted by Børvik et al., ([Børvik, Hanssen, Langseth, & Olovsson, 2009](#_ENREF_7)), the accuracy of uncoupled methods in modelling the response of blast-loaded structures has been demonstrated. They have shown that the Lagrangian (uncoupled) approach, when compared with full-scale experimental test data, remains accurate and leads to consistent results. Research in this paper successfully employs a robust uncoupled analysis method in which blasts loads are estimated separately using a hydrocode and subsequently remapped onto numerical response prediction models.

The computational behaviour of structures under extreme loading conditions such as blast and high velocity impact is predominantly controlled by separation and collision of individual elements. In such cases, the rules of continuum mechanics are no longer applied. The Finite Element Method (FEM) is widely utilized for modelling the behaviour of complex continuum bodies or solids under extreme loading conditions. However, modelling the transient dynamic response of reinforced concrete members, featuring strength degradation due to fracture and fragmentation of concrete, using the FE technique is very challenging. Advanced computational methods such as the Applied Element Method (AEM) ([Meguro & Tagel-Din, 2001](#_ENREF_15)) have been developed as alternatives to FE techniques, and they present a comprehensive understanding of the behaviour of structures subject to extreme loading conditions. Numerical simulations discussed in this paper utilize the AEM-based numerical algorithm “*Extreme Loading for Structures* (*ELS)*” ([Applied Science International [ASI], 2013](#_ENREF_2)), with accurate input blast loads based on CFD simulations conducted using the hydrocode “*Air3D*” ([Rose, 2001](#_ENREF_22)), representing a robust uncoupled approach.

This paper quantifies the effect of transient uplift forces on the vulnerability of internal columns subject to lateral blast pressures in terms of: (i) – column vibration properties, (ii) – ultimate shear capacity and, (iii) - strain histories in main reinforcement. Research presented in this paper forms a part of an on-going research programme developing a set of design and assessment guidelines to prevent column failure following internal building detonations. These design and assessment guidelines fall outside the scope of this paper’s research and will be reported separately when concluded. Section 2 of the paper outlines the research methodology covering high-resolution CFD simulations and numerical modelling of column response. Section 3 compares numerical predictions with archived test data to demonstrate the accuracy of the numerical modelling technique. A parametric study highlighting the effect of transient uplift forces and blast wave confinement on the column’s overall vulnerability to internal building detonations is presented in Section 4. In section 5, conclusions are presented.

1. **Research methodology**

Research in this paper comprises two main stages: (i) – high resolution CFD simulations of internal explosions and subsequent blast wave propagations to obtain pressure histories acting on internal columns; (ii) – high fidelity numerical modelling of column response using the Applied Element technique.

* 1. *CFD simulation of internal building detonations*

One of the main drawbacks associated with CFD modelling relates to spatial discretization requirements in modelling large flow fields using Eulerian cells. Detonation of high explosives and initial shock expansion involve extremely high pressures and temperatures with contact discontinuities ([Rose, 2001](#_ENREF_22)). The cell size within this region of the flow field requires to be sufficiently resolved to capture the accurate behaviour of the detonation products and properties of the expanding blast wave. These highly-dense spatial discretization requirements coupled with the limitation of computational power drastically limit the size of flow fields that is numerically achievable ([Morison, 2007](#_ENREF_16)). In contrast to this, once the shock wave has sufficiently expanded and propagates smoothly in medium-to-low pressure regions, the cell size can be increased with little noticeable influence on the accuracy of results ([Century Dynamics, 2011](#_ENREF_10)). As a result, CFD simulations of explosions and blast wave propagations in large 3-D flow fields can be conducted by varying density of the spatial discretization at different stages of the wave propagation. In CFD simulations conducted in this research, the widely adopted “*remapping technique*” ([Century Dynamics, 2011](#_ENREF_10); [Rose, 2003](#_ENREF_23)) was employed to obtain accurate results over entire 3-D flow fields. This permits the scale of the problem to be maintained within computationally manageable limits. The remapping technique employed in this research is a two-step process. In the first step, detonation of high explosives and subsequent wave propagation until the shock front reaches ground level were modelled. Within this domain, the blast wave is spherically expanding, and the radial distribution of physical properties of the blast wave can be obtained by analysing a wedge filled with air as shown in Figure 1a. Each one-dimensional simulation was terminated just prior to blast wave interaction with the ground surface. In the second step, the data registry at the end of 1-D simulation was directly remapped into the 3-D flow field as illustrated in Figure 1b.

The solution algorithm implemented in Air3D has been proven to be inexpensive in terms of computational resources and retains sufficient accuracy and robustness towards simulating blast phenomenology ([Stephens & Nicholas, 2007](#_ENREF_25)). However, this cannot always assure reliable outputs as overall accuracy is significantly dependant on the properties of spatial discretization within flow fields. For example, if the cell size is unrealistically large compared to the wave length of the shock, numerical predictions tend to be noticeably inconsistent with test results. Conversely, if the cell size is too small, CFD models would become computationally inefficient, and the solution time would be substantially extended without any appreciable improvement in accuracy ([Century Dynamics, 2011](#_ENREF_10)). In order to eliminate errors associated mesh discretization properties of flow fields, a number of CFD simulations with different combinations of cell sizes in 1-D and 3-D flow fields were conducted. Results ([Wijesundara, 2014](#_ENREF_30)) indicated that a spatial discretization achieved by using 0.1mm thick cells for scaled 1-D wedge models, and by using 2.5mm cubic cells for scaled 3-D models can accurately estimate complex pressure histories induced by internal explosions covering the range of shock intensities considered in this investigation.

The majority of reported investigations ([Bao & Li, 2010](#_ENREF_5); [Ngo, Mendis, Gupta, & Ramsay, 2007](#_ENREF_18); [Paramasivam, 2008](#_ENREF_21)) have considered column response due to external explosions that typically associated with large stand-off distances. In case of an internal explosion, the stand-off distance *R*, and the corresponding scaled distance *Z*, to the nearest target would be fairly small due restricted space inside the building; scaled distance is typically in the range of 0.2-1.2 kg/m1/3. Table 1 provides the results of a series of CFD simulations conducted in this research. The rear face peak pressure and total impulse *PRear* and *IRear*, are given as percentages of the corresponding front face blast load parameters (*PFront* and *IFront*), at different scaled distances. The results provided in Table 1 indicate that in particular to blast wave interactions with slender components such as columns, both rear face peak pressure and total impulse are fairly smaller than the corresponding front face blast wave parameter. In consequence, rear face pressures were conservatively neglected in numerical simulations described in this paper.

* 1. *Applied Element modelling of column response*

The Applied Element technique defines a group of normal and shear springs along each surface of 3-D volumetric cube elements to maintain connectivity between elements ([Applied Science International [ASI], 2013](#_ENREF_2); [Tagel-Din, 2009](#_ENREF_26)) . Each spring represents nonlinear path dependant properties of the material being modelled. The defined degrees of freedom represent the rigid body motion of the elements. Although elements behave as rigid bodies, internal deformations within elements are estimated using spring deformations around each element. Relative displacements between two adjacent elements cause stresses in springs that share common element faces.

Numerical models described in this paper comprise four main elements: the core concrete column, adjacent floor plates, cross girders connecting the top end of the column, and an additional virtual column representing the axial stiffness attributed to upper floor members (see Figure 2). Archived results from a number of investigations ([Bao & Li, 2010](#_ENREF_5); [Kusumaningrum, 2010](#_ENREF_12); [Paramasivam, 2008](#_ENREF_21)) indicate that for simplicity, numerical models of r.c. columns are frequently constructed based on simplified boundary conditions (e.g. fixed-fixed). However, the applicability of these constraints towards modelling column response is limited because, depending upon the stiffness of adjacent girders and floor plates, partial translations and rotations can occur at the supports. It is difficult to model these partial translations and rotations in mathematical terms, but are nonetheless important for the accurate representation of the column response. The virtual column representing upper floors comprises a number of rigid plates connected in sequence, by normal and shear springs. In order to limit shear deformations, shear modulus of the concrete in the virtual column was fixed to a significantly large value. The virtual column permits linear-elastic axial deformations governed by a modified Young’s Modulus *Emod*, as described in Eq-1, where *L* and *A* are the height and cross-sectional area of the virtual column respectively. *Kup* is the elastic axial stiffness attributed to upper floor members, which was estimated from supplemental FE analyses (Wijesundara, 2014).

In order to obtain the representative behaviour of r.c. members due to impulsive loading environments, reinforcement details and the interaction of reinforcing bars with the surrounding concrete require to be modelled precisely ([Williams & Williamson, 2011](#_ENREF_31)). In numerical models constructed in this research, structural arrangements of reinforcement were accurately defined using both implicit and explicit methods. Reinforcing bars in floor plates and cross girders were implicitly modelled; representing reasonable simplicity and numerical robustness. In implicit method, each bar was modelled using a set of separate springs (see Figure 3a), the behaviour of which was governed by the non-linear path dependant material properties of steel. All reinforcement in the column was explicitly modelled as unique structural components. A full interface connectivity between reinforcement and concrete matrix was defined using normal and shear springs (see Figure 3b). This interface behaviour is dependent on the properties of normal and shear springs governed by the material properties of concrete. Concrete elements were re-meshed around each reinforcing bar to permit interface material generation. This technique accurately captures the spall behaviour of concrete due to high strain rate loading conditions and leads to a more accurate representation of column behaviour ([Tagel-Din, 2009](#_ENREF_26)). The interface between stirrups and main bars was defined on a bearing only mechanism (with zero tensile strength) to allow accurate contact between stirrups and main bars. Figure 3c illustrates the reinforcement arrangement at the column-to-beam connection; with continuous reinforcing bars maintaining connectivity and realistic boundary conditions.

In order to maintain a proper continuity of longitudinal reinforcement into the upper floor column, a set of highly stiff normal springs connecting the virtual column with the longitudinal reinforcement was introduced. With regard to internal building detonation scenarios (close-in range explosions), uniform blast pressures over the entire column height cannot be reasonably expected and variations of blast pressures along the column are required for a realistic representation of blast wave interaction ([Williamson et al., 2010](#_ENREF_32)). In order to take account of this effect, each column was divided into 10 key segments and average pressure histories were separately applied on to each column segment. These pressure histories were estimated using high-resolution CFD simulations described in section 2-1. This takes account of variations of shock arrival time, peak pressure and total impulse along the column height. Transient uplift forces were applied as pressure loads acting on bottom surfaces of floor plates and cross girders; representing a realistic coupling behaviour of uplift forces with column shear forces. Floor plates and cross girders also consist of static permanent and service loads (typical of office buildings) applied as pressure loads acting downwards.

Figures 4a and 4b show the constitutive models used for steel and concrete respectively. Nonlinear path-dependant material properties of steel was defined using the Ristic material model (see Tagel-Din and Megura, ([2000](#_ENREF_27)) for more details) in which the tangent stiffness of reinforcement was calculated based on spring strain, loading status and previous loading history controlling the Bauschinger’s effect. The Maekawa compression model (see Okamura and Maekawa, ([1991](#_ENREF_20)) for the governing equations) was adopted for modelling concrete under compression. This model uses three parameters to define the envelope of stress-strain curve for concrete: the initial Young’s modulus, compressive plastic strain and the fracture parameter representing the extent of initial damage in concrete. For concrete springs subject to tension, spring stiffness was set to be zero following cracking. Concrete cracking occurs when the principal tension stress reaches the tensile strength of concrete. Referring to Figure 4c, the principal tension stress (σp) and the local crack inclination angle (β) were calculated from Equations 2-4.

|  |  |
| --- | --- |
|  | (2) |
|  | (3) |
|  | (4) |

Sensitivity studies conducted in this investigation indicated that coarser element meshes can be utilized for floor plates and girders. Element meshes comprising approximately 40,000 and 16,000 elements in total were appropriate for the entire floor plate and for each individual beam respectively. A further refined mesh (>25,000 elements) was required for the core column due to explicit reinforcement that required to be accurately connected to the surrounding concrete. In total, each numerical simulation comprised three evaluation stages: non-linear static analysis of the structural system due to permanent load, non-linear static analysis due to compressive axial loads (service loads), and non-linear transient dynamic analysis due to coupled lateral and uplift blast loads.

Blast loads typically produce strain rates in the range of 102 to104 s-1 ([Ngo et al., 2007](#_ENREF_18)). These high strain rate loading conditions noticeably increase the strength of concrete and steel reinforcement. In this paper, Dynamic Increase Factors (DIF) of concrete and steel reinforcement provided in the US military design guidance “*UFC 3-340-02*” ([US Army, 2008](#_ENREF_29)) were referenced. The corresponding values of DIF for the concrete compressive strength and the yield stress of steel were 1.60 and 1.23 respectively. These DIF values were estimated using the lower limit of the strain rate range typical of blasts; a reasonable approach leading to safe conservative numerical predictions. The strain rate formulations integrated into material constitutive models were not of interest due to: (i) – the literature consists of numerous successful numerical modelling examples conducted without direct inclusion of strain rate effects. In a comprehensive investigation conducted by Williams and Williamson ([Williams & Williamson, 2011](#_ENREF_31)), numerical models consisting of rate-dependant material properties have shown to perform with unrealistic stiffness when compared to the expected results on the basis of published data and, (ii) - It has been reported in the literature that directly accounting for strain-rate dependency in modelling concrete response may be inappropriate, as models that do not employ this feature are still able to show rate dependency as stress waves propagate through a finite medium ([Schwer, 2009](#_ENREF_24)). Schwer (2009) discusses an informal theory that explains the strength increase in concrete at high strain rates. This theory suggests that concrete does not exhibit direct strain-rate dependant material behaviour similar to metals, instead; it experiences an effective increase in strength attributable to inertial confinement and the fact that concrete strength is pressure sensitive.

1. **Numerical modelling verification**

Numerical modelling verification consists of two steps: (i) - verification of Air3D simulation of internal building detonations, and (ii) - verification of numerical modelling of column response using the Applied Element technique. In the first step, Air3D simulations of internal explosions were compared with alternative high-resolution CFD simulations conducted using Autodyn. In the second step, accuracy of numerical modelling of column response was demonstrated by comparing numerical predictions with archived test data.

* 1. *Step 01 – Verification of CFD simulations using Air3D*

Two Internal building detonation scenarios were considered: (i) – a vented internal blast inducing resultant unidirectional lateral loads on the internal column (C-5) in the virtual building as illustrated in Figure 5a; and (ii) – a contained internal blast inducing symmetric bidirectional lateral loads on the internal column (see Figure 5b). Figure 6 shows the contour plots of pressure profiles at different time steps corresponding to the vented blast wave propagation modelled using Autodyn. Comparisons of overpressure and impulse histories estimated at a height of 0.7m on the internal column (C-5) and at the centre of the bottom surface of the slab panel 1 [directly above the charge point considered in blast scenario b (see Figure 5b)] are shown in Figure 7. Both overpressure-time and total impulse-time histories estimated from Air3D simulations compare well with corresponding predictions from Autodyn, demonstrating accuracy and robustness of the constructed CFD models.

The presence of strong perimeter walls in blast scenario b led to a noticeable confinement of the blast wave. The pressure histories due to the confined blast consisted of a primary pressure pulse followed by a sequence of secondary pulses due to multiple reflections of the blast wave at perimeter walls (see last two graphs in Figure 7). These secondary pressure pulses can increase the total impulse transferred on to structural members, ultimately reducing the member survivability in confined blast environments.

* 1. *Step 02 – Verification of numerical modelling of column response*

Test results of four r.c. columns have been reported by Nikl ([Nikl, 2006](#_ENREF_19)) and were used to benchmark the numerical modelling procedure of column response. Table 2 provides test details consisting of static axial compression force and blast scenario for each column. Columns C3 and C4 were identical to columns C1 and C2 respectively, and were tested in an explosive loading laboratory to reproduce observed damage in columns C1 and C2 during field tests ([Nikl, 2006](#_ENREF_19)). Columns were 3276mm in clear height and 356mm square in cross section. Characteristic compressive strengths of concrete for columns C1, C2, C3 and C4 were 38.7, 43.9, 39.0 and 41.0MPa respectively. Tensile strength and Young’s modulus of concrete were estimated based on the compressive strength values using codified relationships provided in the BS 8110 ([British Standards Institution [BSI], 1997](#_ENREF_8)) and in the American Code of Practice-ACI 318 ([American Concrete Institute [ACI], 2002](#_ENREF_1)) (see Table 3). Grade 420 reinforcing bar was used in both longitudinal and transverse directions. The yield stress and ultimate strength of steel were 462MPa and 627MPa respectively. Columns were reinforced with eight 25mm (3.2%) longitudinal reinforcing bars and 10mm shear links (with 90 degree bends) at 324mm centres (see Figure 8). The nominal concrete cover was 38mm.

Figure 9 compares the extent of overall damage in column C1. Two numerical models based on connectivity of the column to cross girders at the upper end were analysed. In Case 1, top end vertical translations during static loading stage were permitted, whilst both ends remained fully fixed in the blast loading stage. Although fully-fixed boundary conditions at both ends are widely assumed for simplicity, it can, in some circumstances, lead to artificially stiff numerical models that underestimate column response parameters. In solution, columns in Case 2 were modelled together with adjacent cross girders and floor plates at the floor level to accurately simulate partially-fixed boundary behaviour permitting partial translations and rotations. Predictions from numerical simulations under both cases agree well with test data (see Table 4). Failure modes shown in Figures 9b and 9c demonstrate the capability of numerical models towards accurately capturing the complex failure mechanism of column C1 observed during blast trials as shown in Figure 9a. Peak lateral displacement obtained from numerical simulations for column C1 agrees well with the corresponding value obtained from blast trials (267mm). In Case 1, the peak lateral displacement of 214mm increased to 236mm with introducing more realistic support conditions in Case 2. Similarly, the maximum bending moment of 422.6kNm measured between the bottom and mid-height of the column in Case 1 increased by 11% to 469.4kNm (between the top and mid-height of the column) in Case 2. The maximum support shear at the bottom increased by 6.6% (from 3698.3kN to 3943.5kN) when the top end partial translations and rotations were permitted in Case 2.

With regard to the integrity of the longitudinal and transverse reinforcement in column C1, test observations concluded that the longitudinal bars remained intact (no ultimate tensile failure), whilst the stirrups in the shear critical regions (near the supports) ruptured ([Nikl, 2006](#_ENREF_19)). This conclusion is strongly supported by numerical predictions. Figures 10a and 10b show strain (as a ratio of the ultimate tensile strain, εult) histories in the longitudinal bars and stirrups respectively. Strains were measured in the middle longitudinal bar at the rear face at both ends of the column, and in two ruptured stirrups within the shear critical areas towards either end of the column. The maximum strain attained by the longitudinal bar near the lower end of the column is about 60% of the ultimate strain (0.0512), whereas at the top the corresponding strain was around 30-40% of the ultimate strain, noticeably below the steel rupture strain. In Case 2, a slight reduction in the peak tensile strain (from 0.0208 to 0.0153) in the longitudinal bar is evident when compared with the maximum strain measured in Case 1 (see Figure 10a); attributed to the difference in support conditions between the two analyses cases. Figure 10b shows strain histories in the two stirrups considered. In both stirrups, longitudinal strains were measured in the leg where rupture occurred. At ultimate strain (0.0512), the stirrups failed and the longitudinal strain rapidly dropped to near zero. This is because the interface between the stirrups and surrounding concrete had been severely damaged when rupture occurred, leading to a weakened bond strength limiting transferred stresses from the surrounding concrete.

Column C2 (identical to column C4 tested in laboratory) was tested for a noticeably weaker blast scenario than for columns C1 and C3 (see Table 2). The response of column C2 was shown to be shear- dominant with a peak lateral displacement of 51mm, and with visible diagonal shear cracks followed by slight concrete fragmentation near the supports (see Figure 11). Figures 12a and 12b show strain histories in the longitudinal bars and stirrups in the column. Strain measurements were recorded at identical locations as in column C1. It is evident from Figures 12a and 12b that there is a notable difference in the strain histories between the longitudinal bars and stirrups, characteristic of the state of deformations in steel (elastic or plastic). The ultimate strain in longitudinal bars was smaller than the yield strain (0.0023). Consequently, longitudinal bars responded elastically with no large plastic deformations, leading to an almost constant stiffness (~ elastic stiffness) during both loading and unloading branches. Residual strains associated with these elastic deformations were negligible. In contrast to this observation, the ultimate strain in the considered stirrups was significantly greater than the yield strain, but slightly below the rupture point. This led to plastic strains followed by large residual in the stirrups. In consequence, strain histories in the stirrups were dominated by large plastic and residual strains (see Figure 12b). Numerical predictions in terms of the maximum lateral deformation [58.6mm] (see Figure 12c) and overall damage pattern agreed well with corresponding experimental data, demonstrating accuracy of numerical models towards modelling shear-dominant response behaviour of r.c. columns due to high strain rate loading conditions.

In order to further demonstrate the accuracy of the numerical modelling procedure quantitatively, it was utilized to model the response of three RC beam specimens (B1, B2 and B3) tested in a shock tube by Magnusson ([Magnusson, 2007](#_ENREF_13)). Beams were 300mm×160mm rectangular in cross section and 1500mm in clear span. The reinforcement structural arrangement is shown in Figure 13. Compressive strengths of concrete for beams B1, B2 and B3 were 92MPa, 133MPa and 173MPa respectively. The explosive charge was positioned at a distance of 10m from the concrete beam and in the centre of the tube’s cross section (at mid-height level of beams). The TNT equivalency was 3kg, 3.5kg and 4kg respectively for beams B1, B2 and B3. Numerical models of the beams were constructed using the procedure described in section 2. Figure 14 compares the experimental support shear history for each beam with numerical predictions. Since the numerical predictions of support shear featured an oscillatory response behaviour, a fitted polynomial curve was included for better comparison of the numerical results with test data. It is evident that the numerical predictions in terms of peak shear and initial column stiffness (slope of shear-deflection curves) are in a good agreement with corresponding test data. This level of consistent behaviour of numerical predictions with experimental test data clearly demonstrates the reliability of numerical modelling procedure adopted in this research. In overall, the comparison studies conducted in this section adequately cover both qualitative and quantitative verifications of the adopted numerical modelling procedure reflecting current modelling practice.

1. Numerical modelling application
   1. *Problem definition*

The virtual building geometry considered was a part of a ground floor comprising one internal concrete column surrounded by eight perimeter columns spaced at 4m in either direction with a clear height of 3m. All columns were 300mm square in cross section comprising an identical structural arrangement with material property analogues to the verification study described in section 3.2. Stirrup spacing was reduced to 200mm in accordance with design requirements. The blast scenario was defined at 40kg TNT equivalence detonated at 0.9m above the ground surface at a stand-off distance of 3m from the internal column, producing only unidirectional lateral pressures on the internal column due to the primary blast wave. Although likely figures for the explosive type and quantity could be predicted by considering factors such as socio-political background, type and use of the structure, terrorist tactics, and past events within the region ([Asprone, Jalayer, Prota, & Manfredi, 2010](#_ENREF_3)), the exact point of detonation inside the building cannot be defined with precision. Analyses in this paper were set to the worst case blast scenario in terms of uplift and blast wave reflections, which would represent ultimate limit state for design. To demonstrate the key features of internal column response following detonation, four design scenarios were considered: Case A-Virtual Building with no perimeter walls or frangible walls (vented pressure) and no uplift forces; Case B- Vented pressure condition with uplift forces; Case C- Virtual Building with rigid perimeter walls (contained pressure) and no uplift forces; Case D- Contained pressure condition with uplift forces.

* 1. Discussion of results

Figure 15 shows the extent of damage in the internal column at 53ms from detonation. With no simultaneous uplift forces and the pressure is allowed to vent along the building perimeter (e.g. typical ground floor parking areas) in Case A, the column remained almost intact and the concrete was effective in resisting applied forces. The maximum support rotation was 0.68 degrees, well below 2 degrees (the upper bound value for type 1 sections) within which the concrete cover is thought to be structurally significant and the cover on both surfaces of the column remains intact ([Mays, Smith, & Cormie, 2009](#_ENREF_14)). Introducing uplift tension forces in Case B led to an overall increase in longitudinal tensile strains subsequently causing concrete cracking to a greater extent. In consequence, the integrity of the column within the shear critical regions was significantly disturbed leading to fragmentation of the concrete cover. The corresponding maximum support rotation was 1.43 degrees; slightly above twice the peak support rotation corresponding to Case A. Concrete within the shear critical regions of the column was almost ineffective in resisting induced shear forces due to substantial uplift forces followed by extensive damage near the supports (see Case B in Figure 15). During damage progression observed in Case B, shear resistance attributed to the concrete dropped, and eventually stirrups dominantly resisted the shear forces, a net reduction in the overall shear resistance of the column. Concrete damage near the supports also led to a significant reduction in column stiffness, increasing the peak lateral displacement and period of vibration of the column by almost twice the respective value corresponding to Case A (see Figure 16a). In Figure 16, time *t*, was defined as ratios of the natural period of vibration of the column *T*, estimated to be 18ms. The lateral displacement *d*, and the corresponding horizontal velocity *v*, of the column were defined as ratios of the column effective depth *deff*, and a simulated velocity *vim*, (*deff /T*) respectively.

For rigid perimeter walls introduced along the building perimeter in Cases C and D (e.g. typical basement storage or parking areas surrounded by soil embankments or strong concrete walls), both lateral blast pressures and uplift forces acting on the internal column were noticeably magnified. The internal column was subjected to net lateral loads over an extended period (see Figure 17). Figure 17, in which pressure *P*, and impulse *I*, histories acting at mid-height and near the top end of the column are given, shows that the column was laterally loaded by a sequence of shockwaves. The main blast wave was immediately followed by a secondary wave due to direct reflections of the blast wave at the building perimeter. Over the column mid-height region the secondary wave was, in magnitude, almost identical to the main blast wave (Figure 17b). The secondary blast wave arrived at the front face of the column at approximately 5.9ms from detonation. In this case, the synchronization of arrival of the secondary wave with column response led to a secondary energy transferral before the column reached peak lateral deformations governed by the effect of the primary blast wave (Figure 18a). In consequence, the secondary pressure wave deformed the column further from its initial ultimate equilibrium position, eventually causing plastic deformations in longitudinal reinforcement (see Figures 18b and 18c) followed by significant lateral deformations as shown in Figures 15c and 15d. In Figures 18b and 18c, strains *ε*, in the middle longitudinal bars at the front and rear faces of the column were defined as ratios of the yield strain *εy*.

The response histories shown in Figure 16 indicate that in Case D, the secondary blast wave noticeably influences the column vulnerability because, column strength and stiffness were considerably reduced due to time-variant uplift forces. Consequently, the column could no longer effectively resist subsequent lateral loads induced by secondary pressure waves. The maximum support rotations in the column corresponding to Case C and Case D were 6.92 and 5.14 degrees respectively, both greater than 5 degrees which is the upper bound for type 2 sections within which the concrete cover is thought to be structurally insignificant but may continue to contribute to the inertial resistance of the column ([Mays et al., 2009](#_ENREF_14)). The peak support rotation in Case D was slightly smaller than that in Case C due to the effect of secondary moments induced by uplift tension forces at large lateral deformations.

Figure 19 illustrates the normalized axial force (N) and shear force (F) histories estimated at an effective depth (d) away from the bottom end of the column for both vented and contained blast conditions. Figures 19a-d also show the variation of column overall shear capacity with respect to transient axial force acting in each column. Taking account of the effect of transient axial forces, the overall shear capacity of each column was approximately estimated using the procedure provided in the Eurocode 2 (CEN, 2002). Due to the predicted spallation behaviour of concrete cover in analysis cases B, C and D, only the column depth confined by tension and compression longitudinal reinforcement was considered in shear capacity estimations. When the effect of upward blast pressures was disregarded in analysis cases A and C, the axial force remained nearly constant at the initial static compression force, which was equal to 642kN. Considering this, overall shear capacity of the columns considered in these two analysis cases was assumed to be constant over the entire response domain as shown in Figures 19a and 19c. The corresponding shear capacity is 425.2kN and the peak shear was estimated to be 490.3kN, which is 15% higher than the shear capacity of the column.

Upward blast pressures considered with the analysis cases B and D led to a sudden drop in the static axial compression force eventually casing axial tension forces in the column as shown in Figures 19b and 19d. This increased tension gradually reduced the shear resistance attributable to concrete. At an axial tension force of approximately 383kN, the column shear resistance was mainly governed by stirrups and the overall shear capacity decreased to109kN; 74% drop compared to the overall shear capacity in analysis cases A and C. In analysis case D, stirrups within the shear critical segments towards either ends of the column ruptured leading to an ultimate shear failure.” The axial and shear force histories shown in Figures 19b and 19d indicate that the rate of transient axial forces is considerably smaller than that of column shear forces; particularly due to difference in period of vibrations of the column and the floor system as a whole. In order to obtain a realistic coupling behaviour of transient axial and shear forces, numerical modelling is required to accurately model this axial load transferal behaviour by considering the effect of transient response of the floor system subject to upward blast pressures.

The effect of uplift tension forces on the vulnerability of concrete columns was further demonstrated when strain histories in the longitudinal reinforcement were compared. Figures 20 and 21 show the strain histories in the middle-rear facing and middle-front facing longitudinal bars. Strains were measured at the mid-height of the column. These strain measurements highlight the characteristics of column response either with or without the effect of uplift forces. Following arrival of the primary blast wave, the column initially responded elastically. As the column lateral deformations gradually increased, concrete cracking was intensified with gradual transferral of uplift forces on to the column. As a result, column response rapidly switched into a membrane-dominant response (tensile membrane action) featuring significant longitudinal tensile strains in each longitudinal bar irrespective of the bar location (either rear face or front face of the column). This is evident from the normalized strain histories at the mid-height of the column shown in Figures 20a and 20b. Initial compressive stresses in the longitudinal bar at the mid-height of the column front face gradually reduced and transferred to negative stresses (tensile stresses) at large uplift forces. Conversely, initial tensile stresses in the rear facing bars at column mid-height were further increased by the effect of uplift forces.

Figure 21 shows the corresponding strain histories for analyses Cases C and D. Despite the effect of uplift forces and multiple reflections along the building perimeter, middle-rear facing bars remained elastic at the column mid-height (Figure 21a). Longitudinal reinforcement at the column rear face yielded at the bottom section without the effect of uplift forces. The middle-front facing bar yielded at the column mid-height with or without the effect of uplift forces (Figure 21b). Substantial strains in the longitudinal bars due to the uplift effect led to large relative displacements between the longitudinal bars and surrounding concrete. These large relative deformations significantly influenced the crack distribution in the column causing an intense cracking and deterioration of interface bond between the longitudinal bars and surrounding concrete as illustrated in Figure 22. If the anchorage length provided is inadequate, there is a risk of main longitudinal reinforcement pulling off from the support leading to an unexpected and premature failure of the column; a key failure mechanism observed in some of the civilian buildings damaged during World War II ([Baker, Leader Williams, & Lax, 1948](#_ENREF_4)).

1. Conclusions

Numerical simulations of blast trials conducted utilizing the Applied Element Method accurately reproduced the damage progression and ultimate failure mechanism in reinforced concrete columns, characteristic of local concrete spalling due to high strain rate blast loading conditions. When examining a performance based approach, the numerical modelling technique discussed in this paper would provide a more accurate representation of column performance than simplified approaches leading to economical benefits. Simplified boundary conditions (e.g. Fixed-Fixed), which are widely considered in numerical modelling of column response, can readily lead to underestimations in design parameters depending upon connectivity of the column and stiffness of connecting cross girders. In some circumstances, modelling partial translations and rotations at beam-to column connections would significantly enhance accuracy of numerical predictions. This can be achieved by modelling adjacent floor plates and cross girders along with the core column to precisely maintain connectivity and continuity of the structural system. Modelling adjacent floor plates and cross beams also facilitated load transferral and precise synchronization of uplift forces with lateral blast pressures acting on concrete columns; an important requirement for accurate representation of blast-column interaction following internal building detonations. Depending upon the detonation point with respect to the target, both uplift forces and extended lateral pressure histories due to confined blasts can significantly deteriorate the overall strength and stiffness of reinforced concrete columns. As a result, in-depth investigations and commensurate design recommendations for protective design of internal reinforced concrete columns are important based on numerical modelling techniques that reliably reproduce this complex blast-structure interaction phenomenon. In addition and importantly, there is a high probability that concrete columns under internal blast environments can exhibit an unexpected and premature failure due to substantial uplift forces. This can occur when longitudinal reinforcing bars are inadequately anchored to resist large uplift forces resulting from contained blast pressures. Conventional design practice for r.c. columns only considers axial compression forces, with no reliable mechanism to account for tensile forces. In solution, anchorage and continuity of the longitudinal reinforcement throughout the column height should be adequately maintained to ensure key structural resilience.

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