Dynamic Progressive Collapse of an RC Assemblage Induced by Contact Detonation

Jun Yu\textsuperscript{a,b}, Tassilo Rinder\textsuperscript{a}, Alexander Stolz\textsuperscript{a,1}, Kang-Hai Tan\textsuperscript{b}, Werner Riedel\textsuperscript{a}

\textsuperscript{a}Dept. Safety Technology and Protective Structures, Fraunhofer Institute for High-Speed Dynamics, Ernst-Mach-Institut, Am Klingelberg 1, D-79588 Efringen-Kirchen, Germany

\textsuperscript{b}Protection Technology and Research Center, School of Civil and Environmental Engineering, Nanyang Technological University, 50 Nanyang Avenue, Singapore 639798

\section*{Abstract}
The nature of progressive collapse is a dynamic event caused by accidental or intentional extraordinary loading. Most published experimental programs are conducted statically, without consideration of the accidental loading and regarding progressive collapse as event-independent. This paper demonstrates the more realistic process of progressive collapse in an experimental program on reinforced concrete sub-assemblages collapsed by a combination of dead weight loading and contact detonation. The dynamic results are represented systematically at different aspects and compared with previous published quasi-static experiments in terms of structural mechanisms, crack patterns and local failure modes. Moreover, the dynamic increase factor (DIF) of reinforcing bars and the dynamic load amplification factor (DLAF) are investigated and discussed. Following the above comparisons and the findings in the dynamic tests, previous quasi-static test results can be linked to actual progressive collapse behavior more convincingly. Finally, the dynamic tests also highlight the effect of contact detonation on structures, which are often not considered in quasi-static tests and design guidelines. The test results indicate that contact detonation cause the uplift and the out-of-plane actions to the sub-assemblage before their downward movement under gravity load, in which the strain rate of reinforcement is between $10^{-2}$/s to $10^{-1}$/s. Moreover, the structural mechanisms are similar in both quasi-static and dynamic tests.

\textit{Keywords:} Progressive collapse, detonation, reinforced concrete, dynamic increase factor, dynamic load amplification factor, compressive arch action, catenary action

\section{Introduction}
The partial collapse of Ronan Point apartment in London in 1968, the severe damage on Murrah Federal Building in Oklahoma City in 1995 due to terrorist attack and the 9/11 WTC event in 2001 sequentially raise public concerns and intensify research interests on progressive collapse of buildings. The design methodology in current building guidelines (DoD 2010; GSA 2003) and codes (ASCE 2005; EN 1996-1-7 2006) against progressive collapse is broadly divided into indirect

\textsuperscript{1}corresponding author: stolz@emi.fraunhofer.de
and direct methods. The indirect method specifies prescriptive ‘tying force’ provisions which are deemed sufficient for avoiding progressive collapse. Depending on whether the damage in columns is allowed, the direct method is further categorized into alternate load path (ALP) approach and specific load resistance method. ALP method is applied by removing one column at each time and check the structural resistance of remaining buildings through structural analysis, including linear static, nonlinear static or nonlinear dynamic analysis (NLD). Since progressive collapse essentially involves material nonlinearity and dynamic effects, NLD analysis is the most accurate. However, NLD analysis using Finite-Element-Methods is so computationally-demanding and time-consuming (Marjanishvili 2004) so that practicing engineers are reluctant to use it. As an alternative, “Applied Element Methods (AEM)” for efficient NLD collapse simulation are being developed and are recently available in commercial software (Helmy et al. 2012; Tagel-Din 2009), but this approach has not widespread in day-to-day building design. Therefore, static analysis is still by far the most preferred approach, but the results should be modified by linear increase factor (LIF) and dynamic increase factor (DIF), respectively, to relate linear and nonlinear static procedures to NLD procedures (DoD 2010). To avoid confusing the DIF for material strength in dynamic loading, the DIF for structural capacity is replaced by dynamic load amplification factor (DLAF) in this paper, further explained in section 4.

To validate design methods and investigate structural performance of reinforced concrete (RC) frames at large deformations, many quasi-static tests on RC frames or sub-assemblages have been carried out under column removal scenarios (Orton et al. 2009; Sadek et al. 2011; Sasani and Kropelnicki 2008; Su et al. 2009; Yi et al. 2008; Yu and Tan 2011; 2012). The experimental results show that with adequate horizontal restraints, RC frames can develop compressive arch action and catenary action, of which structural capacities greatly exceed flexural capacities. Otherwise, structural resistance is mainly dominated by flexure. Izzuddin et al. (2008) have proposed a framework to convert available nonlinear static structural behavior into dynamic capacities based on energy equilibrium. Furthermore, the dynamic performances of small-scaled RC frames under sudden removal of corner columns (Kai and Li 2012) and steel frames under sudden removal of middle columns (Liu et al. 2012) are reported, providing valuable information on structural dynamic performance. However, all the aforementioned tests decoupled the initial damage of columns and the structural performance of remaining structures, which may not represent real situations in the actual accident or event. Two in-situ tests on RC buildings with explosively removed columns are reported in the literature (Sasani et al. 2007; Sasani and Sagiroglu 2008), but the maximum deflections of the beams above removed columns in the two tests were only 6.6 mm and 6.1 mm, respectively. They are not representative for structural behavior under large deformations.

In this paper, therefore, a dynamic test with large dead load and with explosive removal of the middle column was conducted to investigate:

(1) the difference between event-independent tests and the blast test
(2) structural dynamic behavior up to a large deformation
(3) whether high strain rates are induced to increase material strength
(4) the dynamic load amplification factor DLAF.

In particular, the specimen detailing was the same as in previous detailed static investigations of the same author (Yu and Tan 2011) in order to highlight the differences in dynamic response.
2 Test Preparations

2.1 Specimen Design

To focus on structural transient dynamic behavior, three RC beam-column sub-assemblage specimens were cast at Fraunhofer EMI, Germany, following the specimen design in Yu and Tan’s tests (2011). Each specimen consisted of two single-bay beams, two end-column stubs and one middle column. Due to symmetry, the geometric dimensions and the detailing of one-half specimen are shown in Fig. 1. The beam section was 150 mm by 250 mm with a concrete cover of 20 mm. The middle column stub section was 250 mm square. The column stubs were 450 mm wide and 400 mm deep, so that they provide sufficient anchorage for the longitudinal bars in the beams. The top reinforcement ratios by area at the middle joint and the end stub interfaces were 0.73% (3T10), and one top bar was curtailed at the location 925 mm away from these interfaces. The bottom reinforcement ratios by area were 0.49% (2T10) throughout the entire specimen, and the bottom bars were lap spliced in the middle joint with the splice length equal to 410 mm according to ACI 318-05. Note that “T” represents ribbed reinforcement with a nominal yield strength of 500 MPa. All stirrups were designed as two-legged R6 hoops with a 135° hook, and were distributed with a center-to-center spacing of 100 mm. “R” stands for round reinforcement with a nominal yield strength of 500 MPa. Three specimens were named as SD-1, SD-2 and SD-3, respectively, in which SD represents sub-assemblages under dynamic tests.

![Fig. 1: Specimen detailing, displayed on the left side of the symmetric sample (Unit: mm)](image)

2.2 Test Configuration and Instrumentation

Fig. 2 shows the test set-up for specimen SD-1. During the test, each end-column stub of the sub-assemblage was seated onto a steel roller and anchored into a heavy concrete ring with four horizontal bolts. Two bolts were in the top layer and the other two in the bottom layer. The distance between the two layers was 600 mm. The concrete ring, with a self-weight of 130 kN, was seated on the ground. The middle column was supported by bricks on the ground. On top of the middle column, a constant dead weight of 47 kN contributed by two concrete blocks and a steel transfer frame was applied. An explosive charge was placed on top of the bricks and near the bottom of the middle column. Before the charge was detonated, the constant dead weight was transferred to the ground directly through the middle column and the brick support. After the charge was detonated, the bricks and the bottom end of the middle column were explosively removed, and the dead weight of 47 kN was redistributed dynamically to the beam-column sub-assemblage. Shortly (around 100 ms) after the contact detonation on the column, the middle joint started moving downwards.
Following the test on specimen SD-1, the test set-up for specimens SD-2 and SD-3 were modified by anchoring the two concrete rings onto the ground with two steel angles for each, as illustrated in the inset in Fig. 2. Moreover, the dead load applied to the specimen SD-2 was reduced to 27 kN by using two smaller concrete blocks, in order to stay below the failure limit for derivation of the DLAF. In summary, the differences for three tests are listed in Table 1.

Table 1: Summary of load and boundary conditions in the three tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Applied dead load (kN)</th>
<th>Boundary conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD-1</td>
<td>47</td>
<td>Two concrete rings \textit{seated} on the ground</td>
</tr>
<tr>
<td>SD-2</td>
<td>27</td>
<td>Two concrete rings \textit{anchored} onto the ground</td>
</tr>
<tr>
<td>SD-3</td>
<td>47</td>
<td>Two concrete rings \textit{anchored} onto the ground</td>
</tr>
</tbody>
</table>

A high-frequency data acquisition system with a sampling rate of 1 MHz was applied in the experimental program. The vertical deflection at the middle joint was measured by a laser for specimen SD-1 and by a linear variable differential transducer (LVDT) for the remaining two tests. An accelerometer was installed on the middle joint of specimens SD-2 and SD-3. Each horizontal restraint bolt was mounted with two strain gages, as indicated in Fig. 2. Through the measured strains, the horizontal reaction forces were calculated in section 3.5. Prior to the casting of
specimens, 20 strain gages were attached onto steel reinforcing bars. During the test, the variations of bar strains at critical beam sections were recorded to shed light on the force transfer mechanism within the specimen.

2.3 Material Properties of the Structural Specimens

All specimens were cured more than 28 days. Immediately following the structural tests, three 150 mm concrete cubes were tested to obtain the concrete strength for each specimen. The cube compressive strengths were 36 / 41 / 50 MPa for specimens SD-1 to 3, respectively (cylinder strengths 26 / 31 / 40 MPa according to (EN 1992-1-1 2004)).

Uniaxial tensile tests were conducted on four reinforcing bar samples with gage length of 200 mm to determine the material properties of reinforcing bars used in three sub-assemblage specimens. The yield and the tensile strengths were 528 and 607 MPa, respectively, based on the nominal cross-section area. The technical strain at ultimate tensile strength was 15.4%.

3 Experimental Results

Specimen SD-1 is used in the following for demonstrating testing procedure. The explosion of the charge occurred at 0 ms, as shown in Fig. 3(a). Fig. 3(b) indicates that the detonation induced an upward shock pulse in the air and the column. After the explosive removal of the middle column and its brick support, the dead load of 47 kN was redirected to the beam-column sub-assemblage, as what would happen in a real building. Following the shock wave stage, the specimen started moving downwards, as indicated in Fig. 3(c). In the test on specimen SD-1, prior to the middle column hitting the ground, the structural resistance failed to capture its downward movement. That is, the energy absorbed by the specimen was less than the external work done by the dead load, and the difference between the internal and external work was transformed into the kinetic energy of the downward movement. Eventually, the two concrete block weight also touched on the ground, as shown in Fig. 3(d).

Fig. 3: Testing procedure with lateral dead weight in front of the charge and middle column; for charge position see Figs. 2 and 14(a)
With improved boundary conditions and a smaller applied dead load, the downward movement of specimen SD-2 was arrested by the structural resistance during the redirection process of loads. However, the load of 47 kN still caused specimen SD-3 to move down to contact with the ground. Detailed test results for three specimens will be illustrated in the following subsections.

3.1 Contact Detonation Damage of the three Middle Columns

To allow the sub-assemblage specimens to move down in a larger free space, the charge size was increased by a factor of 1.52 for specimens SD-2 and SD-3 so as to damage a larger part of the middle columns. Due to contact detonation, the free end of the middle column became cone-shaped. In the concrete cover and at the four corners of columns sections more concrete spalled off than in the confined concrete core, as shown in Fig. 4. Hairline cracks perpendicular to the column axis were observed due to shock wave propagation along the middle column. Table 2 summarizes the damage lengths of the middle columns in concrete core and cover, which were determined by deducting the respective residual lengths (measured at the column side facing the explosive charge and denoted as $a$ and $b$ in Fig. 4(a)) from the original lengths. Obviously, a larger charge damaged more concrete.

![Fig. 4: Damage of middle columns after contact detonation and dead weight loading](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Charge</th>
<th>Damaged length at detonation face (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD-1</td>
<td>M</td>
<td>115~185</td>
</tr>
<tr>
<td>SD-2</td>
<td>1.52M</td>
<td>122~280</td>
</tr>
<tr>
<td>SD-3</td>
<td>1.52M</td>
<td>130~230</td>
</tr>
</tbody>
</table>

3.2 Dynamic Middle Joint Displacement

Fig. 5(a) shows that directly after the detonation the middle joint of specimen SD-1 moved upwards within the first milliseconds with a displacement of 2.5 mm before moving downwards after 100 ms. Between 5 to 90 ms the laser displacement gage was temporally ineffective due to the interference of the fire ball and the fragments of a wooden plate, as shown in Fig. 3(b). At around 100 ms, the middle joint displacement (MJD) was 16 mm downwards. At around 504 ms, the damaged middle column hit the ground. The eventual MJD was around 450 mm.
Fig. 5: Time resolved middle joint displacements. SD-1 with partial information due to optical interference with the laser gages, SD-2/3 with undisturbed LVDT. Initial uplift (10ms) and oscillations by the shock induced in the column and downward movement starting after 100ms.

In the test on specimen SD-2, the laser was replaced by an LVDT to measure the MJD to avoid optical interference by the explosion. Similar to SD-1, the middle joint of specimen SD-2 moved
upwards up to 30 mm within about 10 ms after the detonation, as shown in Fig. 5(b). The shock wave propagating in the middle column caused the middle joint to oscillate vertically. After the shock waves had decayed at around 80 ms, specimen SD-2 started moving downwards under the applied load of 27 kN. The movement was arrested with a maximum displacement of 54.1 mm at 302 ms. Thereafter, the free vibration of SD-2 was decaying with a permanent displacement of around 50 mm.

In the test on specimen SD-3, the middle joint initially moved upwards within 10 ms after the detonation. Beyond 100 ms, the center of the specimen then permanently moved downwards under the applied load of 47 kN until the specimen hit the ground with a maximum displacement of 471.7 mm at about 525 ms, as shown in Fig. 5(c).

3.3 Crack Patterns and Comparisons with Static Tests

![Failure mode of specimen SD-1 in dynamic tests](image)

(a) Failure mode of specimen SD-1 in dynamic tests

![Crack patterns of specimen SD-3 with equivalent deformation mechanisms as in SD-1](image)

(b) Crack patterns of specimen SD-3 with equivalent deformation mechanisms as in SD-1

![Failure mode of specimen with non-seismic detailing in static tests](image)

(c) Failure mode of specimen with non-seismic detailing in static tests (Yu and Tan 2011)

Fig. 6: Comparisons of failure modes in dynamic and static tests indicates very similar deformation mechanism because of identical detailing

After the full dynamic response, Specimens SD-1 and SD-3 eventually failed to sustain the load of 47 kN prior to hitting the ground with the crack patterns shown in Figs. 6(a) and (b), respectively. The overall crack patterns in two specimens were similar, but the cracks and concrete spalling at the beam ends connected to end columns stubs of Specimen SD-3 were more severe than those in SD-1, indicating the mobilization of a larger bending moment at compressive arch action stage in
specimen SD-3 due to improved boundary conditions. Moreover, concrete crushing was observed in the region near cutting-off point in Specimen SD-3, as highlighted in Fig. 6(b).

A comparison between Figs. 6(a) and (c) shows that the both the overall and local crack patterns in the dynamic test were very similar to the static test (Yu and Tan 2011). Note that the specimen used for comparison had the same detailing. Tensile cracks, penetrating the entire beam sections, were extensively distributed along the beam spans, indicating the development of beam axial tension. Moreover, the beam-column connections near the end-column stub functioned as conventional plastic hinges with flexural cracks and crushed concrete. Two concentrated cracks occurred around the free ends of lap-spliced bottom bars in the middle joint. At one side, two bottom bars fractured during the test, suggesting that one development length for tension splice is adequate for continuity.

After the removal of the middle column, specimen SD-2 successfully sustained a load of 27 kN with the maximum dynamic deflection of 54 mm. Fig. 7 shows the crack patterns at the end of the test. At the beam segments near the end-column stubs, only flexural cracks were observed without evident concrete crushing. Near the middle joint, several flexural cracks occurred and penetrated the entire beam sections. These were attributed to the up and down movements associated with negative and positive bending moments. Besides, no obvious concrete crushing was observed.

![Image](image_url)

Fig. 7: Crack patterns of specimen SD-2 at the end of test

### 3.4 Acceleration Measurements on the Middle Joint

An accelerometer was installed at the middle joint side opposite to the charge. The distance of the top free end of the middle column to the charge was 1.37 m, and the sound velocity in concrete is around 3600 m/s, so that the shock propagation in the middle column induced a longitudinal oscillation at about 2.6 kHz. With an Eigen-frequency of 54 kHz, the accelerometer was fast enough to derive the dynamic response of the specimens. The maximum acceleration during the shock loading exceeded the measuring capacity of ± 500g, but this was not the primary focus of the investigation. Therefore, the accelerometer readings are not quantitative with respect to amplitudes, but they provide the timing of events during the test. The signals of the accelerometer on specimen SD-3 indicated three sequential events in Fig. 8: Following the detonation, the shock wave induced extremely large acceleration within the first 6 ms (Fig. 8(a)). Between 287~291ms the bottom reinforcing bars at one side of the middle joint fractured. Thereafter, the large variation of accelerations corresponded to the middle column hitting the ground.
3.5 Dynamic Horizontal Reaction Forces

As illustrated in Fig. 2, the strains of the horizontal restraint bars were recorded during the test. The measured strains were then used to determine the corresponding reaction force provided by each bar. Fig. 9 shows the time histories of horizontal reaction forces of specimens SD-2 and SD-3. The horizontal reaction was equal to the sum of reactions along the four restraint bars. The negative and positive values correspond to the development of axial compression and tension in the two-bay beam, respectively (Yu and Tan 2011).

In specimen SD-2, only beam axial compression was mobilized during the test, indicating that the structural mechanism to redistribute the load was compressive arch action (CAA). The maximum negative horizontal reaction was around 55.04 kN at the time of 221 ms. In specimen SD-3, both beam axial compression and tension were mobilized, indicating the development of CAA and catenary action. However, Fig. 9 shows that the main structural mechanism in specimen SD-3 was still CAA. At 443.5 ms, the beam axial tension was mobilized, indicating the onset of catenary action, but the development of catenary action was terminated due to the contact of the residual middle column with the ground.
The relationship of horizontal reaction force and the middle joint displacement (MJD) in specimen SD-3 are shown in Fig. 10(a), which indicates that catenary action was mobilized at a MJD of 357 mm. To compare the differences in dynamic and static performances of RC sub-assemblages, the variation of horizontal reaction force of Specimen S2 in static tests (Yu and Tan 2011) is added in Fig. 10(b). It should be repeated that both static and dynamic specimens had the same detailing as shown in Fig. 1.

**Fig. 10:** Variations of horizontal reaction forces in tests: negative and positive values correspond to beam axial compression and tension, respectively.
A comparison between Figs. 10(a) and (b) indicates that similar to the static test, both CAA (associated with axial compression) and catenary action (associated with axial tension) were sequentially mobilized in the dynamic test on specimen SD-3. However, the peak horizontal reaction force at CAA stage in the dynamic test was smaller than that in the static test, respectively. At CAA stage, provided that the same boundary conditions were achieved in both quasi-static and dynamic tests, due to load amplification effect in dynamic tests, a smaller axial compression of SD-3 in the dynamic test could be equivalent to a larger axial compression of S2 in the quasi-static test. Moreover, the boundary conditions in the dynamic tests were actually weaker than those in the quasi-static tests, further reducing the maximum axial compression. At catenary action stage, adequate space was provided in the quasi-static test to allow Specimen S2 to deform to its limit so that large beam axial tension can be mobilized. However, the development of catenary action of Specimen SD-3 in the dynamic test was terminated due to the residual column hitting the ground. At that moment, beam axial tension was still relatively small. This situation is very possible under explosive removal scenarios in the real world.

3.6 Shock Uplift, Torque and Structural Movement at the Beam-Column Interfaces

Bar strains of RC sub-assemblages were extensively studied and explained in the quasi-static tests (Yu and Tan 2011; 2012). However, to find out the differences between event-independent tests and the dynamic tests under blast loading, the strain Gage readings in specimen SD-1 are systematically analyzed in the following. Due to similarity, the strain gage readings in the other two specimens are not shown in this paper.

The layout of strain Gages within the RC sub-assemblage is shown in Fig. 11. The sections near the middle joint interfaces (Section 3 and 4), the end-column stub interfaces (Section 1 and 6) and at the mid-span of the single-bay beams (denoted as Section 2 and 5) were selected as critical sections. At each critical section, the strains at both top and bottom bars were measured. All recordings are displayed at a small (0-50ms) and a large time scale (0-1s) to investigate the different dynamic loading domains of shock/stress waves and dynamic structural response, respectively.

![Fig. 11: Layout of strain gages in all samples with critical sections close to middle and end columns and at mid-span of the beams.](image-url)
Fig. 12 shows the time histories of bar strains at the middle joint interfaces (sections 3 and 4). These two sections were nearest to the explosive charge throughout the entire beams. It can be observed in Fig. 12(a) that following the detonation, the oscillations of the strains were severe due to shock and stress waves. Moreover, in the shock wave stage, the top bar (indicated as gage 3a) was in tension and the bottom bar (denoted as gage 3c) in compression, indicating a negative bending moment at the middle joint region. This phenomenon can be more clearly observed in Fig. 12(c). In contrast, the middle joint in the static tests was subjected to positive bending moment directly after applying load (Yu and Tan 2011). The occurrence of negative bending moment between 2 and 30 ms in Fig. 12(c) suggests that the middle joint sustained a large upward force due to the initial upward acceleration of the middle joint, as shown in Fig. 5. However, during the subsequent slower structural response, tensile strains in the bottom bars exceeded the top bars and yielded first (at 160 ms), and the top bar reversed to compression, as shown in Figs. 12(b) and (d). These clearly pointed out the development of a positive bending moment while the specimen was moving down due to the gravity loads. The timing agrees very well with the dynamic middle joint displacement shown in Fig. 5.

Strain gages 3c and 4c were mounted on the side of the middle joint, which was laterally loaded by the contact charge. The detonation of the eccentric charge caused not only in-plane action but also out-of-plane action, including bending moment and torque, to the two-bay beam. Therefore, strain gages 3b and 4b experienced earlier and larger tension than gages 3c and 4c, respectively, as shown in Figs. 12(a) and (c), and this trend remained until the yielding of two bottom bars occurred. At
around 150ms, two bottom bars at sections 3 and 4 yielded, as a sharp spike indicated in Figs. 12(b) and (d). After the yielding, the strain gages were spoiled, so the drop of tensile strains in gages 3b, 3c, 4b and 4c (indicated as dash thinner lines in Figs. 12(b) and (d)) could not reflect the actual situations. Moreover, the bottom bars fractured at the same side of section 3, resulting in a sudden reduction of strain in the top bar at around 330 ms, as shown in Fig. 12(b).

![Fig. 13: Time histories of bar strains at end-column stub interfaces of SD-1: Positive bending moment induced by shock-driven uplift of the middle joints up to around 15 ms, negative bending and top bar yielding in the following structural response beyond 100 ms. One bottom bar in tension and the other in compression caused by out-of-plane actions](image)

Fig. 13 shows the evolution of bar strains at the end-column stub interfaces. Fig. 13(a) illustrates that following the detonation, two bottom bars were in tension and the top bar in compression, indicating a positive bending moment at the end-column stub interface. However, the static counterpart was subjected to a negative bending moment right from the beginning of the loading (Yu and Tan 2011). The positive bending moment can also be found at section 6 prior to the time of around 10 ms. This phenomenon confirmed that the specimen was subjected to an upward force due to the upward acceleration of the middle column induced by shock. Besides, Figs. 13(a) and (c) show that at the bottom layer of reinforcement, gages 1c and 6c experienced tension whereas gages 1b and 6b sustained compression between 10 and 50 ms. This was attributed to out-of-plane action caused by the eccentric charge. Figs. 13(b) and (d) demonstrate that after around 100 ms, gages 1c and 6c worked together with gages 1b and 6b in compression, respectively. After around 20 ms, the tension in top bars became more dominant in the entire beam section, indicating the development of negative bending moment. At around 250 ms, corresponding to a MJD of about 150mm (see Fig. 5(a)), the top bars at both sections 1 and 6 yielded. The following Gage readings after the peak
tensile yield strains (indicated as dash thinner lines in Figs. 13(b) and (d)) may not reflect the real conditions due to subsequently damaged strain gage.

In summary, the bending moments of the beam-column interfaces in X-Y plane from the shock-driven upward movement within the first 20 ms, as shown in Fig. 14(a), was opposite to those at the progressive collapse stage between 100-500 ms, as illustrated in Fig. 14(b). Therefore, the top bars in the middle joint region sustained tension initially followed by compression, and the top bars at the end-column stub interfaces underwent compression initially and then tension. Moreover, the explosion-induced torque and out-of-plane (i.e. X-Z plane) bending moment on the two-bay beam, as elucidated in Fig. 14(a), resulted in tensile strains of 3b and 4b greater than 3c and 4c at the early stage, respectively. Similarly, the tensile strains of 1c and 6c exceeded those of 1b and 6b, respectively, at the shock-driven stage. It is worthwhile to note that the static tests could only simulate the progressive collapse stage.

![Sketches of bending moment diagrams in dynamic test for the shock driven uplift and out-of-plane bending (or torque) on the middle column and the later downward structural response due to dead weight load](image)

**Fig. 14:** Sketches of bending moment diagrams in dynamic test for the shock driven uplift and out-of-plane bending (or torque) on the middle column and the later downward structural response due to dead weight load

### 3.7 Dynamic Catenary Action at Mid-Span Bars of the Single-bay Beams

Sections 2 and 5 were located near the contra-flexure points of both in-plane and out-of-plane bending moment diagrams, that is, the bending moments at these two sections were very small. Therefore, only small strains developed at sections 2 and 5 prior to around 325 ms, as shown in Fig. 15. At the time of around 325 ms, the tensile strains at the two top bars soared, suggesting the development of catenary action. Moreover, the top reinforcement was not continuous. At sections 2 and 5, there were only two top bars in contrast to three top bars at the beam-column joint and end-column-stub interfaces. Under the same axial tension throughout the entire beam, the strains of top bars at sections 2 and 5 increased much faster than at the sections with three top bars. Finally, a comparison of strain histories in Figs. 15(a) and (b) indicates the symmetry in behavior at sections 2
and 5. After the shock wave stage, the bars at the same reinforcement layer show similar readings, reflecting that the specimen was in in-plane action only.

![Graph of bar strains at mid-span of a single-bay beam in SD-1](image)

(a) At section 2    (b) At section 5

Fig. 15: Time histories of bar strains at mid-span of a single-bay beam in SD-1

4. Dynamic Strength Increase and Load Amplification Factor

4.1 Strain Rates and Strength Increase

The strain rates in the structure during a realistic, detonation driven collapse is one of main interests of this study. High strain rate results in a larger yield strength of bars (Malvar 1998) and an increase in concrete strength, further enhancing structural resistance. It is found that the strains of bars at the middle joint interfaces and the beam end column stub interfaces changed rapidly, and local failure (such as severe cracking and bar fracture) occurred there. Fig. 16(a) shows the strain rate at the middle joint interface. Following the detonation, the strain rate suddenly increased to around 0.3~0.4/s in the first 20 ms. During the gravity-driven downward movement of the specimen, the average strain rates remained around 0.03/s, and the maximum strain rates reached 0.1/s. After detonation, the average strain rates of bars at the end-column-stub interfaces were around 0.01/s and the maximum strain rates reached 0.05/s, as shown in Fig. 16(b).

Fig. 16(c) and (d) show the strain rates at sections 2 and 5 (i.e. the mid-span of the single-bay beams) of specimen SD-1, respectively. The variations of strain rates show similar patterns. Between 300 ms and 600 ms, when catenary action and touching of the ground occurred, sections 2 and 5 were fully stressed and the strain rates of bars ranged from 0.005/s to 0.17/s. In contrast, the strain rates in static test were less than 10^{-4}/s.
Strength enhancement with strain rate is expressed in the form of a dynamic increase factor (DIF), defined as the ratio of the dynamic to static yield (or ultimate) strength. Malvar (1998) has proposed a simple equation to estimate the DIF for both yield and ultimate strength of reinforcement, given as follows:

\[
DIF = \left( \frac{\dot{\varepsilon}}{10^{-4}} \right)^{\alpha} \tag{1}
\]

where \(\dot{\varepsilon}\) is strain rate (1/sec), for yield strength, \(\alpha = \alpha_{fy}\), which is given by

\[
\alpha_{fy} = 0.074 - 0.040 \frac{f_y}{414} \tag{2}
\]

and for ultimate strength, \(\alpha = \alpha_{fu}\), which is given by

\[
\alpha_{fu} = 0.019 - 0.009 \frac{f_y}{414} \tag{3}
\]

where \(f_y\) is bar yield strength (MPa).

In this dynamic test, the yield strength \(f_y\) of reinforcement was 528 MPa, and the strain rates of bars at all sections mainly ranged from \(10^{-2}\)/s to \(10^{-1}\)/s when these sections were fully stressed. Therefore, based on Eqs.(1) to (2), the corresponding DIF for the yield strength of reinforcement ranged from 1.112 to 1.172. All strain Gages attached to the reinforcing bars worked well from the elastic stage...
to yielding plateau, but were not designed to function in the strain hardening stage of bars. Therefore, strain rates of the bars approaching the ultimate tensile strength were not available in test on SD-1. If the strain rates are assumed as $10^2$/s to $10^3$/s, the DIF for the ultimate tensile strength will vary between 1.035 and 1.053. The analyses on specimens SD-2 and SD-3 allow the same conclusions. Since the DIFs in material strengths are not significant, it is not necessary to incorporate them into the structural analysis. Neglecting strain rate dependent strength increase in progressive collapse analysis will result in slightly conservative predictions.

4.2 Discussions of the Dynamic Load Amplification Factor

The dynamic load amplification factor (DLAF) is defined as a ratio of loads which result in the same deformation of a structure in nonlinear static (NLS) analysis and nonlinear dynamic (NLD) analysis. By using DLAF, NLS analysis procedure can be more conveniently used for assessing progressive collapse of buildings with consideration of dynamic effects. Therefore, the empirical formula to calculate DLFA of RC frames is adopted in UFC 4-023-03 (DoD 2010). It should be pointed out that the formula applies only when the structural mechanism is conventional flexure. In contrast, Izzuddin et al. (2008) proposed a more general framework to convert available nonlinear static structural behavior - which are resistance functions from the point view of structural dynamics - into dynamic capacities based on energy equilibrium.

To obtain the nonlinear static structural behavior of the sub-assemblages, numerical simulation was conducted with commercial finite element software Engineer’s Studio. Fig. 17 shows the numerical model. Rigid elements were used to model two supporting concrete rings, which were anchored into the ground. The two-bay beam in a sub-assemblage was modeled with fiber-based beam elements, and the middle joint was modeled with a component-based joint element. At the middle joint interfaces, the springs are set as zero-length. A displacement load was applied at the top of the middle joint. Detailed description of the joint element and the validation of the numerical model is given in the paper (Yu and Tan 2011).

Fig. 17: Numerical model for sub-assemblage specimens, used for obtaining the nonlinear static load-deflection relationship (i.e. resistance function) so as to evaluate the dynamic load amplification factor

Since it is difficult to evaluate the effectiveness of the anchorage of concrete rings to the ground, the boundary restraints on two concrete rings are assumed as fully fixed or laterally restrained with a stiffness $K_d$. Fig. 18 shows the nonlinear static load-deflection history. At a MJD of 54 mm, the static resistance is $P_1=50.42$ kN, and the corresponding DLAF is equal to $50.42/27=1.87$. This can be considered as the upper limit of DLAF in this scenario. Since the anchorage of concrete to the ground could be weaker than fully fixed, the static resistance at a MJD of 54 mm will be less than 50.42 kN. For example, if two concrete rings are assumed as laterally restrained with a stiffness of $K_d=1\times10^5$ kN/m and $5\times10^4$ kN/m, the corresponding structural resistances are 45.06 kN and 40.44
kN at a MJD of 54 mm, respectively. As a result, the DLAFs are 1.67 and 1.50, respectively. However, for three different resistance functions, as shown in Fig. 18, if DLAF is evaluated through energy equilibrium proposed by Izzuddin et al. (2008), which does not consider the initial damage of the remaining structures at the beginning of the progressive collapse stage following the removal of a column (say, due to contact detonation herein), the corresponding values are 1.16, 1.33 and 1.30 for cases 1-3, respectively. The differences of DLAF by the two approaches result from that the contact detonation actually caused initial damages on the two-bay beam through uplifting and out-of-plane actions, but the resistance functions are derived without considering these effects. In other words, ignoring the effect of contact detonation on the two-bay beam leads to overestimated structural resistance, and a large DLAF is obtained. On the other hand, using the energy equilibrium approach without considering the initial damage of the two-bay beams will underestimate DLAF and further overestimate progressive collapse resistance, which is on the unsafe side of structural design. For a multi-story building, the damage from the initial events mainly apply to the story with the occurrence of accidental or extreme loadings, but the remaining stories approximately work in event-independent scenarios.

On the other hand, Fig. 18 indicates that at a MJD of 472 mm, which is the ultimate displacement of specimen SD-3 under dynamic test, catenary action resistance is only 43.25 kN for the three cases, less than the applied load of 47 kN. Therefore, to experimentally determine the value of DLAF, more space to allow deflection should be provided. However, in a realistic loading scenario with a segment of a concrete column removed by detonation, it is likely that a clean and complete removal of the column, as in a static sub-assemblage test, will not occur. Realistic lengths of explosively removed column sections were analyzed as 115 to 130 mm in Fig 4 and Table 2. In such a situation, the concept of DLAF will reach its limit of validity.

5. Key Findings and Suggestions for Further Studies

Three reinforced concrete beam-column sub-assemblages were tested to study the realistic dynamic response of structures subjected to progressive collapse under contact detonation. The specimens were designed with the same detailing, but the applied dead load and the boundary conditions were
different. Furthermore the same detailing was used as in the static tests (Yu and Tan 2011) to allow direct comparison.

The displacement and rebar strain measurements clearly show that a shock-driven acceleration initially uplifted the middle column above the explosive charge in the first 2-20 ms. The measurements also highlight out-of-plane bending moment and torsion acting at the beam sections in the same timeframe. After the shock wave stage, the sub-assemblage started falling down due to applied gravity loads in the time range of 100 – 500ms.

Subjected to a large dead load of 47 kN, compressive arch action (CAA) and catenary action were sequentially mobilized in the dynamic downward motion of specimens SD-1 and SD-3, associated with the development of beam axial compression and tension, respectively. These two assemblages moved down until the residual middle columns hit the ground. However, subjected to a smaller dead load of 27 kN, the downward movement of specimen SD-2 was arrested by CAA at a relatively small deflection (around 50 mm). The crack patterns and local failure modes, such as bar fracture and concrete crushing, were very similar to those observed in quasi-static tests (Yu and Tan 2011).

During the moving down of the sub-assemblages, the strain rates of reinforcing bars in the beams ranged from $10^{-2}$/s to $10^{1}$/s, slightly enhancing the yield strength of bars by 10% to 17%. Therefore, the dynamic increase factor (DIF) of material strength is small or can be conservatively ignored for column removal scenarios.

Through a combined experimental and numerical approach the dynamic load amplification factor (DLAF) in specimen SD-2 under 27 kN was analyzed with an upper bound of 1.86. The high DLAF resulted from that the contact detonation actually caused initial damages on the two-bay beam through uplifting and out-of-plane actions, but the resistance functions are derived without considering any initial damage on the two-bay beam. Therefore, using the energy equilibrium approach without considering the initial damage will underestimate DLAF and further overestimate progressive collapse resistance, which is on the unsafe side of structural design.

In the current tests, the development of catenary action was terminated due to the residual column hitting the ground. This situation is realistic under explosive removal scenarios in the real world, but compromises the analysis of the dynamic load amplification factor (DLAF) in the catenary action stage. Therefore, if the research interest is to find out the DLAF at catenary action stage, the middle columns should be designed such that a much greater length of it could be removed.

During the tests, the horizontal restraint bars were subjected to both axial force and bending moments. Therefore, to obtain the horizontal reaction accurately, at least two strain gages should be symmetrically mounted at one bar section.

Laser displacement measurements are sensitive to the fire ball and ensuing dust cloud. Alternatively, other displacement transducers with high Eigen-frequencies can be used to avoid the noises from the resonance of the transducers. In addition, accelerometers should be installed directly on a specimen rather than on other auxiliary parts attached to the specimen.
Acknowledgements

The work in this paper was part of the project AURIS, funded by the German Federal Ministry of Education and Research and financially supported by the Protective Technology Research Center of Nanyang Technological University, Singapore.

References


