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Explosive loading of multi storey RC buildings: Dynamic response and progressive collapse

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Abstract. The resilience of a city confronted with a terrorist bomb attack is the background of the paper. The resilience strongly depends on vital infrastructure and the physical protection of people. The protection buildings provide in case of an external explosion is one of the important elements in safety assessment. Besides the aspect of protection, buildings facilitate and enable many functions, e.g., offices, data storage, -handling and -transfer, energy supply, banks, shopping malls etc. When a building is damaged, the loss of functions is directly related to the location, amount of damage and the damage level. At TNO Defence, Security and Safety methods are developed to quantify the resilience of city infrastructure systems (Weerheijm *et al*. 2007b). In this framework, the dynamic response, damage levels and residual bearing capacity of multi-storey RC buildings is studied. The current paper addresses the aspects of dynamic response and progressive collapse, as well as the proposed method to relate the structural damage to a volume-damage parameter, which can be linked to the loss of functionality. After a general introduction to the research programme and progressive collapse, the study of the dynamic response and damage due to blast loading for a single RC element is described. Shock tube experiments on plates are used as a reference to study the possibilities of engineering methods and an explicit finite element code to quantify the response and residual bearing capacity. Next the dynamic response and progressive collapse of a multi storey RC building is studied numerically, using a number of models. Conclusions are drawn on the ability to predict initial blast damage and progressive collapse. Finally the link between the structural damage of a building and its loss of functionality is described, which is essential input for the envisaged method to quantify the resilience of city infrastructure.

Keywords: structural dynamics; buildings; explosions; concrete; rate dependency; finite elements, experiments; resilience of infrastructure; damage assessment; safety.

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1. Introduction: Progressive collapse

Progressive collapse is a chain reaction of failures, following the damage of a relatively small

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portion of a structure. This can be due to an accidental explosion, e.g. gas pipe, or be man-made, e.g., terrorist attack. One of the clearest examples of progressive collapse is The Ronan Point apartment building in London, a 22-storey building, constructed in the nineteen sixties. Due to poor design, collapse was triggered by an explosion in one of the building floors. Another example of progressive collapse is the Murrah Federal Building in Oklahoma City. In 1995 a truck loaded with explosives was detonated next to the building, leading to progressive collapse.

Progressive collapse may occur due to a lack of redundancy of the supporting structures. When one part of the structure fails, there are no alternate load paths to redistribute the load, causing a failure chain (domino effect), similar to that of a "card stack". A lack of structural continuity may also lead to progressive collapse, e.g., the Ronan Point building. A classification of the typology of progressive collapse is given in (Starossek 2007).

To avoid progressive collapse, two design approaches exist: (i) indirect design: which relies on the use of tie forces ("catenary action"); (ii) direct design, which is based on the "alternate path" (redundancy). Adequate detailing of RC structures is necessary to avoid progressive collapse (continuous top and bottom reinforcements, close spacing of stirrups, etc.). Reference (Ellingwood *et al.* 2006) contains the "best practices" to reduce the likelihood of progressive collapse.

In recent years, much attention has been paid to the study of progressive collapse, mainly triggered by the increase in the terrorist threat. A bomb attack may cause severe damage and local failure of load bearing, structural elements. Depending on the damage level and the building redundancy, progressive collapse may occur. In (Izzuddin *et al.* 2008, Vlassis *et al.* 2008) progressive collapse of a multi storey building has been studied by the sudden removal of a column. In (Kaewkulchai *et al.* 2004) a beam model is used for the prediction of dynamic progressive collapse. It emphasises the need of dynamic analyses, as opposed to static analysis, which are not conservative. In (Luccioni *et al.* 2004), a dynamic analysis of progressive collapse of a RC building is performed using a hydrocode.

In the method to quantify the resilience of the city infrastructure that TNO develops, the prediction of explosion damage to buildings is the kernel. Local damage is acceptable if it does not extend in time and does not trigger progressive collapse. The structural response under explosive loading is highly dynamic. It involves material nonlinearity (damage, plasticity) and geometric nonlinearity (large deflections), dynamic impact of falling floors, fragments, etc. Altogether, progressive collapse is a very complex phenomenon, which requires solid knowledge of the physics involved and of the computational tools to describe them.

To develop the *resilience method* and select the right computational tools for dynamic structural response, damage, and progressive collapse prediction, it was decided to start with a detailed study of a structural element: a plate (see section 2). Using the gained knowledge at element level, next the response of a multi-storey RC building is studied. The nonlinear explicit finite element code LS-DYNA was selected as a tool. Explicit codes are very suitable to simulate structural transient dynamic problems, such as the response of a building under blast, and progressive failure (implicit codes are more dedicated to model slow dynamics and pseudo-static problems). Aiming at keeping the resilience assessment method as simple as possible but still covering all dominant and decisive mechanisms, the building has been modelled in various ways. A simple 2D beam smeared RC model is studied, followed by a 3D beam-shell model. Due to its simplicity, structural models (beam, shells) are very attractive from an engineering point of view and are the envisaged application in the resilience method. To examine the consequences of the *beam-shell* representation, finally a more advanced solid-rebar 2D model is studied. The case study of the dynamic response

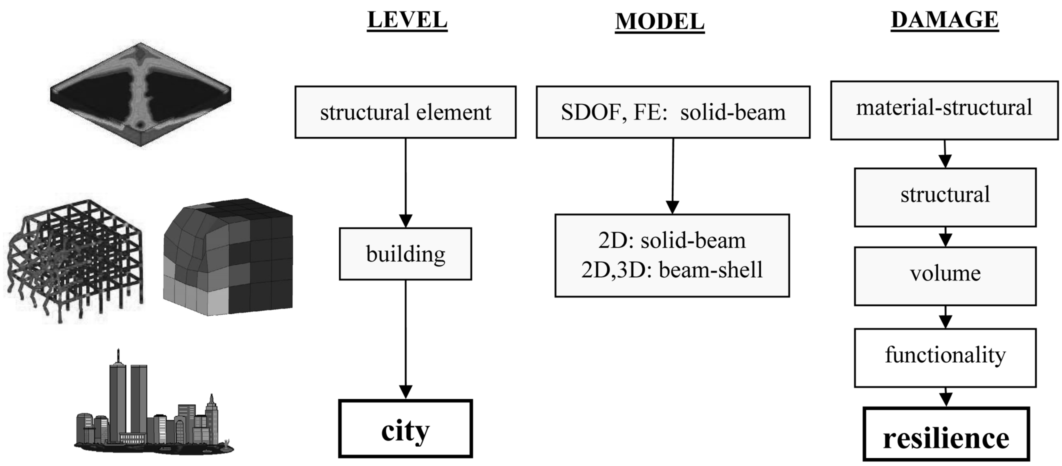


Fig. 1 Bottom-up approach towards the resilience of a city (Coloured parts are the subject in the current paper)

and progressive collapse is given in section 3.

The research showed the necessity to consider the dynamic response of the whole building, because it responds as an integral system and cannot be represented as a sum of individual elements. The method discussed in this paper is able to predict the final structural damage and residual bearing capacity. The next step in the overall programme on the resilience of the built infrastructure is to link the damage and reduction in structural integrity to the loss of functionality of the building. In section 4 a method is proposed to relate the structural damage to a damage parameter that can be linked to the loss of functionality. This bottom-up approach has been sketched in Fig. 1, with the level of study, the model used and the damage type, to achieve the final goal, i.e., determining the resilience of a city.

2. Dynamic response of RC-elements

To quantify the resilience of urban systems to bomb attacks and define enhancement measures, the damage to structures due to explosion has to be determined. Within the TNO research programme, a number of techniques to predict the damage to buildings are assessed according their predicting capabilities. For this application, the *right tool* does not have to be the *scientifically most correct* tool, but it has to cover all dominant phenomena. To start with, the damage of a single RC-element, a square RC-panel, under blast is investigated. Engineering tools, experiments and advanced numerical techniques are compared; see also (Weerheijm *et al.* 2007a). The approach and results are described in this section.

2.1 Engineering response model

In engineering practice, the response of a structure is often studied by means of a SDOF system. The underlying assumption is that there is one dominant deformation mode, from the initial elastic regime up to final failure. Rate effects on material properties are neglected or taken as a constant for the entire response process. A SDOF is governed by the balance equation, Eq. (1)

$$K_{lm} \cdot M \cdot \ddot{u} + R(u) = F(t) \qquad (1)$$

which is widely discussed in textbooks (e.g., Biggs 1964). The load mass factor K_{lm} is derived from the equivalence of kinetic- and deformation energy of the continuum and SDOF system, while the resistance to deformation is represented in the function $R(u)$. Usually, the selected degree of freedom is the mid-span deflection, u. In the SDOF the static resistance function is applied. To judge applicability of the approach for damage assessment, a test procedure has been developed to determine this function of RC-slabs under dynamic loading, see section 2.2. The deformation capacity under blast loads is obtained and the direct comparison with the (theoretical) static resistance function tells us about the validity of the simplifications on the single dominant deformation mode and the constant material properties for the entire response process.

2.2 Reference tests

The resistance of a concrete element to dynamic loading strongly depends on the dynamic response of the structure and the material rate effects. At TNO-DSS a test procedure was developed to derive the dynamic resistance from experiments (Doormaal *et al.* 1996). This method is applied on a batch of square reinforced concrete plates of dimensions 1600 × 1600 × 100 mm, tested in a blast simulator ('shock tube'), at different pressure levels. The plates are simply supported at four sides. To allow for edge rotations, rubber strips are placed between the plates and the blast simulator. Displacements, accelerations and pressure history are recorded using sensors placed at characteristic points on each plate (Fig. 2), which will allow determining the dynamic plate resistance $R(u)$ in Eq. (1). The tests resulted in a reference database for numerical simulations. Detailed information is available of the response and the deformation shape as a function of time. The tests are reported in (Steen van der 2007). The applicability of the SDOF was analyzed; see conclusions in section 2.5.

2.3 The finite element model

The dynamic response of the concrete plates is simulated with the explicit FE code LS-DYNA (Hallquist 2005). Simply supported boundary conditions are assumed at all four sides. Because of the symmetry, only one quarter of the plate is simulated. Concrete is modelled using continuum 8-

Fig. 2 (left) concrete slab in blast simulator, (right) extensive cracking after a blast load of 160 kPa peak pressure

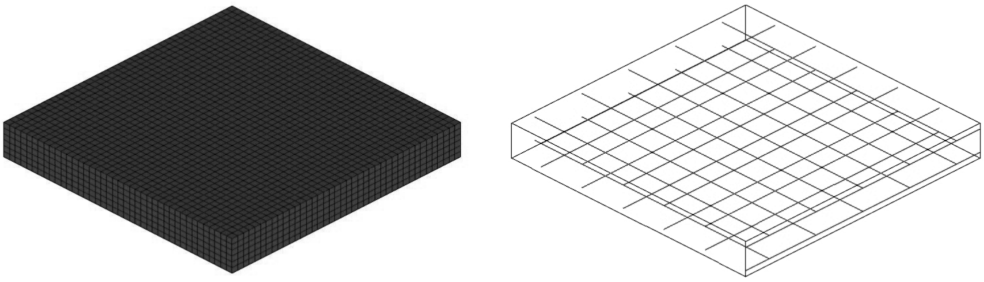


Fig. 3 (left) half mesh with double solid elements (concrete) and (right) rebars

node brick elements (6 layers of elements in slab thickness) and material type K&C (see Fig. 3 left). The K&C model (Malvar 1997, Mediavilla 2007b) is originally based on the pseudo-tensor model (MAT_16) and features a three surface plasticity formulation, softening, shear dilatancy, a non-associative flow rule and strain rate effect. The reinforcement bars are modelled using Hughes-Liu beam elements (Fig. 3-right) and a von-Mises elastoplastic material, with kinematic hardening, strain rate effects and a strain failure criterion. Perfect bonding (no slippage) is assumed between concrete and rebars. The blast pressure and the vertical load are applied on zero-thickness shell elements, attached to the solids.

The resistance-displacement curve is a property of the structure, and therefore should be the same regardless of the magnitude of the loading, provided that the same mechanisms are mobilized and strain rate effects are neglected (see SDOF assumptions, section 2.1). To elucidate this point, the resistance-displacement curve has been computed for an idealized linear decaying blast pressure with a duration of 45 ms and different blast pressures (75-160 kPa), similar to the experiments. After a short elastic phase, the plate evolves from hardening to softening. See Fig. 4. As expected, all curves lie on top of each other. The apparent decrease in stiffness during unloading with increase in blast pressure is due to the increase in plasticity. Note that the K&C model is a softening plasticity model; hence, unlike damage models, there is no degradation of the elastic stiffness. The strain rate effect is noticed in the higher resistance with increasing pressure (i.e., increasing strain rate), see Fig. 5.

The resistance curves and the deformation-time records of the tests and the FE-calculations have been compared (Weerheijm *et al.* 2007b). To study the applicability of the material model and

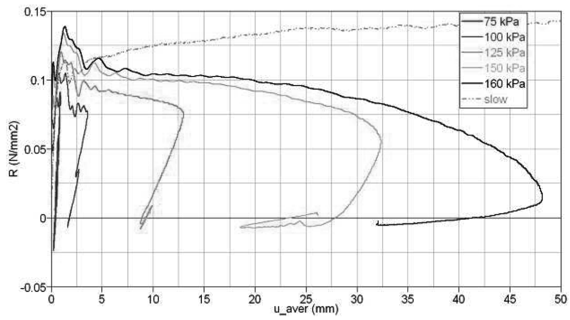


Fig. 4 Resistance-average displacement curves of reinforced concrete plate for blast pressures of 75-160 kPa (45 ms duration)

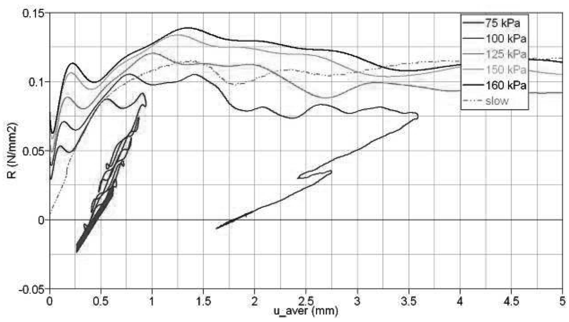


Fig. 5 Close-up of resistance-average displacement

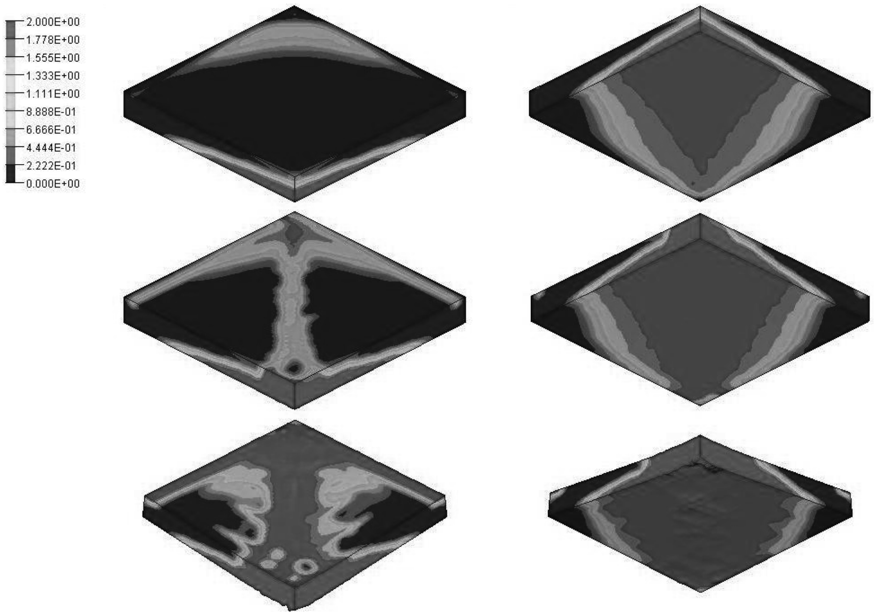


Fig. 6 Damage fields at t = 100 ms, for three different blast pressures 75 kPa, 100 kPa and 150 kPa (45 ms duration); (left) pressure side and (right) back side

model parameters in more detail the inverse modeling technique can be used. A limited study has been performed and reported in (Mediavilla *et al.* 2007c), where the Kalman filter technique was applied. The comparisons showed that the applied schematization and material model are suitable to simulate the overall dynamic response up to failure. Comparison with the tests showed that the accuracy of damage prediction at high pressure loads, i.e., high loading rates and extensive damage, is limited. Note, that material research and data for these conditions is still limited. For recent developments see (Vegt *et al.* 2007, 2008).

To illustrate the predicted damage fields, Fig. 6 shows the *damage parameter* for three blast loads (75, 100 and 150 kPa, all 45 ms duration). The damage parameter in the K&C concrete model is represented by means of a parameter δ, which is given by

$$\delta = \frac{2\lambda}{\lambda + \lambda_m} \tag{2}$$

where λ and λ_m are the modified plastic strain and the modified plastic strain at the point of maximum strength. δ ranges between 0 to 2. The calculated damage fields resemble the crack pattern observed in the experiments. It should be noted that in the experiments the yield line pattern, formed by the zone of macro cracks, is narrow banded compared to the damage zones in the computations. It should also be noted that the value of the damage parameter reflects the maximum damage occurred in tension and or compression. Especially in dynamics, tension and compression stresses alternate and initial damage due to tension has minor importance when the compressive state becomes dominant. This damage parameter is not an objective measure for the final damage stage, while strains are more meaningful.

The resistance curves were obtained from the FE model results, and the agreement with the experiments is good. See Fig. 7.

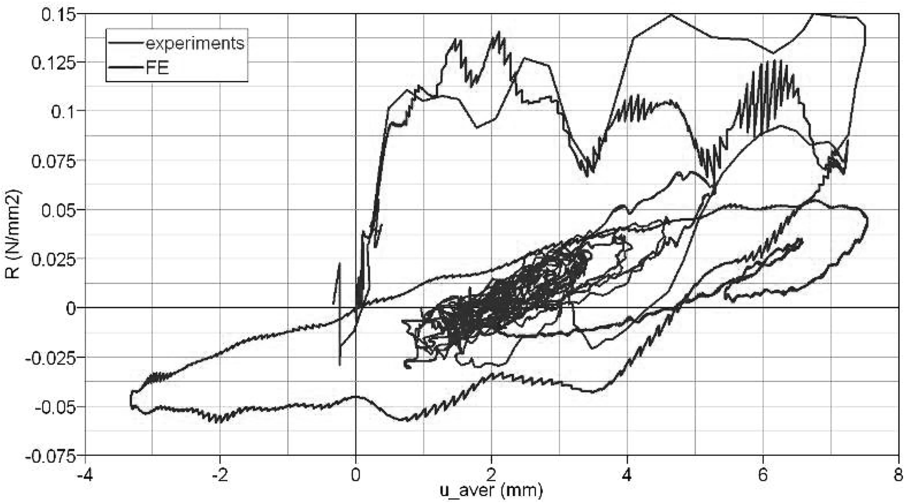


Fig. 7 Resistance curves. (green line) experimental; (brown line) numerical

2.4 Residual bearing capacity

From a safety point of view, it is crucial to determine the plate residual strength after the blast explosion. To examine and illustrate the possibilities of the LS-DYNA finite element code, the following case was explored. A plate carrying a load $q_0 = 2$ N/mm^2 in its plane direction is exposed to a blast pressure, with different amplitudes, a duration of 45 ms and a linear decay (see Fig. 8).

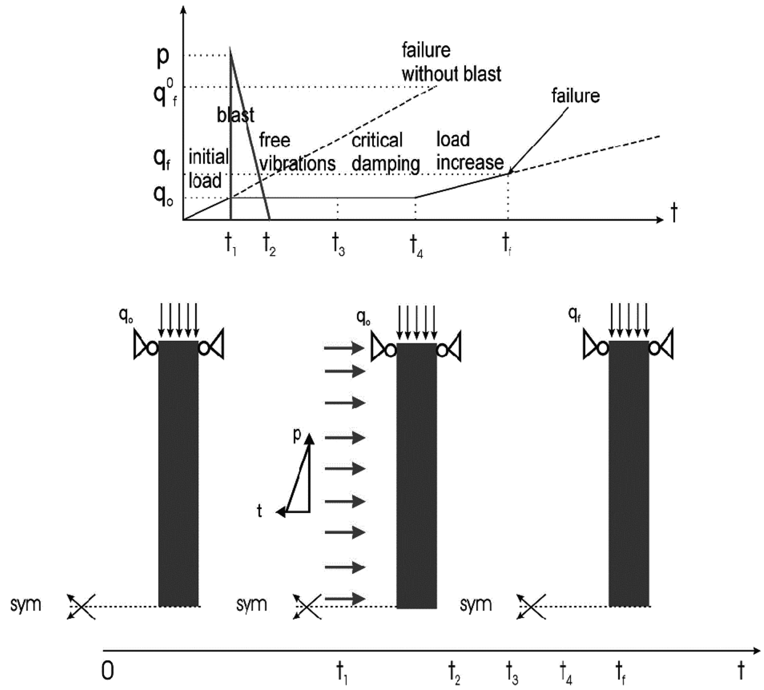


Fig. 8 Loading history to failure strength for a plate under static in-plane load and dynamic bending load

During the preload, free vibrations have damped out using critical damping. During the blast loading, the plate vibrates freely. Then critical damping is again switched on, and the vertical load is increased monotonically, until failure occurs. This determines the residual load bearing capacity. Following this procedure the whole sequence of preloading, blast load, damage development and finally static loading up to failure could be modelled in a stable way. Note that experimental validation of the static residual strength has not been performed yet and should be determined.

2.5 Summary and discussion of RC-element analysis

This section summarizes the results and observations from the *single element-research.* Not all aspects in this summary were addressed in the previous sections, but they can be found in the references (Weerheijm *et al.* 2007a, Mediavilla *et al.* 2007a, 2007b).

- The traditional SDOF approach based on a single deformation mode and midspan deflection is not suitable for determining the full dynamic response and damage development in shock tube tests due to the contribution of higher response modes. Therefore, an equivalent SDOF based on the average displacement was defined and proved to be applicable. The corresponding resistance function was determined for the test slabs and, together with the experimentally derived function, used as reference for the numerical FE-prediction.
- The LS-DYNA (FE) code and the K&C concrete material model were selected to predict damage development and residual strength.
- A modelling procedure was developed and applied successfully to predict blast damage and residual bearing capacity. The numerically predicted resistance corresponds with the experimentally derived relation. The unloading phase is not well represented.
- Modelling the rate effects in concrete and steel bonding needs attention, in order to quantify the damage and residual stiffness more accurately for high amplitude, high rate loading conditions.

The question is whether modelling with beam and shell elements is sufficient as input for the envisaged method to quantify the resilience of built city infrastructure. These possibilities have been examined using a schematised RC multi storey building as benchmark. This part of the research is presented in the next section.

3. Case study

To study the possibilities to predict and describe the dynamic response and damage development in multi-storey buildings due to external blast loading sufficiently accurate, but also as simple as possible, the progressive collapse of a case of a 4-storey RC building has been investigated. The building has a frame structure, composed of beams and columns, and two-way massive RC floors (see Fig. 9).

The distance between the columns is 4 m. The ceilings are 3 m high, except the bottom storey, which is 4 m. The beams and columns have dimensions 0.4 m × 0.4 m, and reinforcement as given in Fig. 9. The floors are 0.15 m thick and have the same amount of reinforcement (0.56%) in both directions. B25 concrete and FeB500 rebar steel is used throughout the structure. All floors are loaded by a uniform dead load of 8.3 kPa. The explosive charge is located on the ground, at 35 m from the centre of the façade. Three different charge weights are studied: 100, 500 and 1000 kg of TNT.

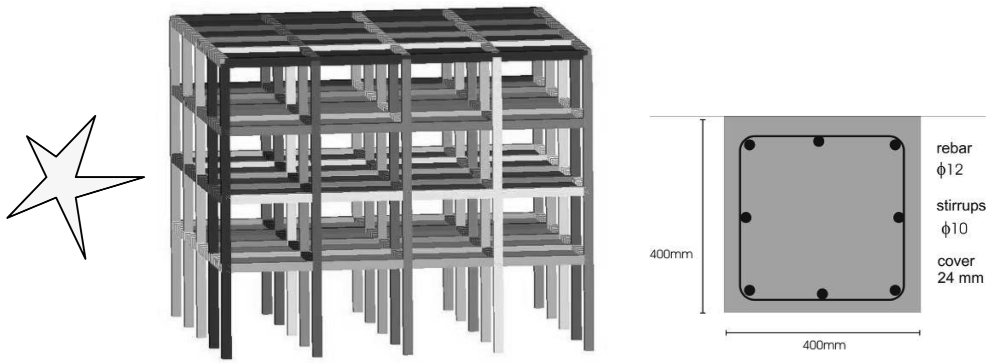


Fig. 9 (left) RC building under blast explosion; (right) cross section of columns and beams with reinforcement

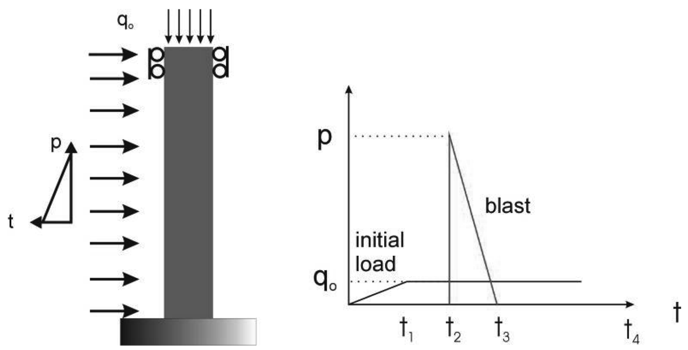


Fig. 10 Loading history

3.1 2D RC Beam model

First, a 2D representation of the building is modelled in LS-DYNA, which corresponds to a 4 m section of the actual 3D building. The dead load is applied gradually in a time period of 0.2 s (t_1) using a ramp function, which is kept constant afterwards (see Fig. 10). Similar to the method developed for the single element in section 2.4, critical damping is applied in order to avoid dynamic effects during this preloading phase, as well as to damp out the free vibrations after the blast is gone. All beams and columns are discretized using ten Hughes-Liu beam elements. The displacements of all nodes are constrained in the out of plane direction. The ground floor nodes are constrained in all directions.

The reflected peak pressure, the impulse and the blast duration t_f have been determined using the ConWep code (US Army 1991) and represented in Table 1, for the three explosive charges. A triangular pressure-time distribution is assumed. The blast pressure is uniformly distributed along the right façade. Note that this is the full reflected load. Due to the 2D-approach, clearing effects cannot be taken into account properly and has therefore completely been neglected. Another consequence of the 2D-approach is neglection of the side pressure that would act on the building. It should also be noted that in real situations of city infra the loading conditions are far more complex (Doormaal *et al.* 2003). The blast load is not uniform. Due to the finite dimensions of the building and the presence of other buildings and objects, numerous reflections and rarefactions will affect the

Table 1 Blast loads

Charge (kg TNT)	Distance (m)	Scaled distance (m/kg$^{1/3}$)	Pressure (kPa)	Impulse (kPa*ms)	t_f (ms)
100	35	7.5	48.5	372.3	15.36
500	35	4.4	130.8	1146.8	17.54
1000	35	3.5	223	1877.5	16.84

blast load on the building. Engineering models and numerical CFD tools have been developed to quantify the blast load (Smith *et al.* 2002). At TNO these tools are available. In this paper a simplified, schematised blast loading is used because the focus is on the structural response and to illustrate the integrated approach.

The RC-structure is modelled with material model **MAT_CONCRETE_EC2 (MAT_172)*. This material model is based on eurocode-2, and describes reinforced concrete as a smeared combination of concrete and steel. Such a smeared approach is computationally very efficient, hence allowing to model large structures, even an entire building. The analyses were performed on a PC using four AMD Opteron 280 processors (2400 MHz), running LS-DYNA double precision on LINUX (64 bits). The computing time was 2 minutes.

MAT_172 is a softening material model. This means, that no additional failure criterion is then needed. Two known limitations of this model are:

(i) it does not take strain rate hardening into account, which is very important under high strain rates, such as during a blast loading. Therefore, in the analyses a dynamic increase factor has been used for both concrete ($f_{ck,\,dyn}$ = 30 MPa) and steel ($f_{yk,\,dyn}$ = 600 MPa).

(ii) it does not incorporate an erosion criterion, which is needed in order to properly model rupture. The fully softened elements hence remain attached to the rest of the structure, with their inertia.

To give an idea of the amount of damage, the plastic strain fields have been plotted in Fig. 11, for the three explosive charges. The 100 kg charge case shows little damage. In the 500 and 1000 kg charge cases, damage is mainly found on the columns of the blast side, as well as the column-beam joints. The 1000 kg charge case is characteristic of progressive collapse, the severe local damage at ground level leads to failure up to the upper floor at the front side of the building. Because the damage occurs mainly in the first bay while the whole front façade is loaded by the blast, one might dispute whether this example is a pure "progressive collapse case" or not.

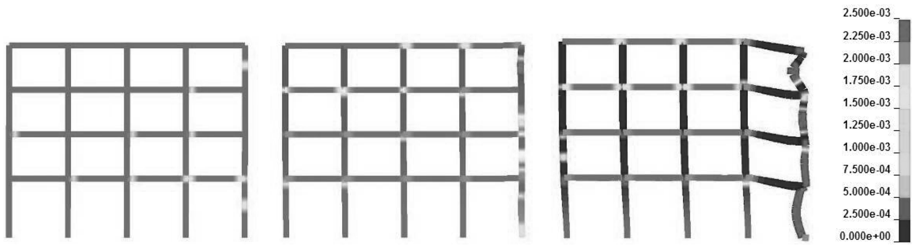


Fig. 11 Equivalent plastic strain plots for different blast charges (stand-off distance = 35 m): (above-left) 100 kg TNT; (above-right) 500 kg TNT; (below) 1000 kg TNT

3.2 History variables 2D RC-model

To analyse the dynamic response in more detail, the bending moment and axial force history of some of the most critical columns, see Fig. 12, have been plotted in Fig. 13, for the 1000 kg TNT charge.

The explosion causes very large bending moments and axial forces on the building foundation. Especially, the blast loaded column (element 361) undergoes a very large compressive force, followed by tensile loading, and failure due to softening. More information of the failure state of these structural elements can be obtained by looking at their M-N trajectories and their position with

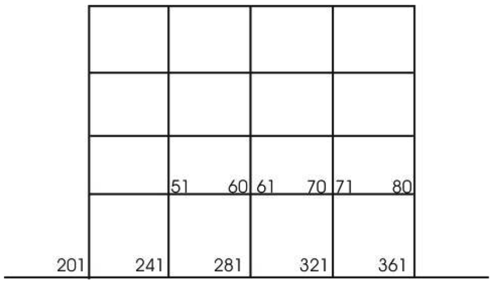
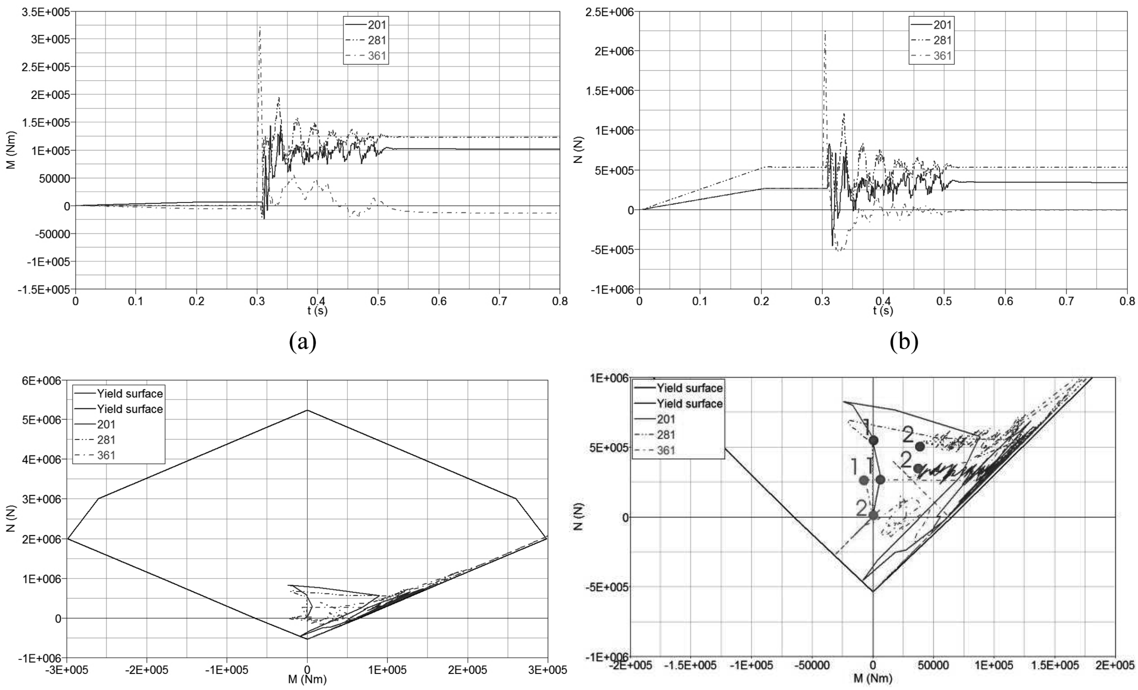


Fig. 12 Studied beam elements



(a) (b)

(c) (d)

Fig. 13 Columns history variables. 1000 kg TNT, at 35 m standoff distance. (a) moment M history; (b) axial force N history; (c) M-N trajectory and failure surface; (d) detailed plot, the (1) and (2) points denote the pre-blast state and the end state respectively

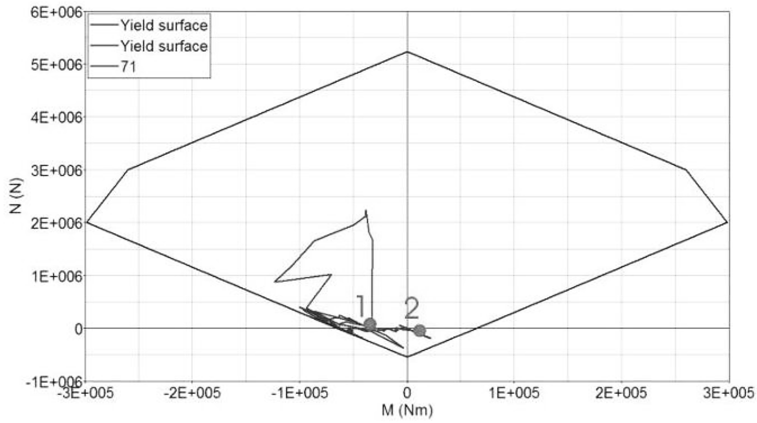


Fig. 14 M-N paths and yield surface of beam 71. The (1) and (2) points denote the pre-blast state and the end state respectively

respect to the M-N failure envelope. Before the explosion, the columns are well in the safe part (overdimensioned) as the (1) dots show. The explosion causes a large increase in bending, which makes the columns reach their maximum capacity. The blast loaded column fails, and the remaining columns must carry an additional load. The final state is represented by the (2) dots in the M-N diagram.

Fig. 14 shows the M-N trajectory of beam 71. The blast load is directly transmitted from the façade to the floors, which explains the large compressive force. The beam eventually reaches the yield surface, and fails due to softening. The results appear to be consistent and illustrate the possibilities of the selected approach to study the static, dynamic and residual phase of the building.

3.3 Static versus dynamic analysis

Progressive collapse is sometimes modelled in literature, by removal of the column most exposed to and heavily damaged by the blast pressure, followed by a pseudo-static analysis. To prove that such an approach is not conservative enough, i.e., unsafe, a pseudo-static analysis has been compared with the transient analysis performed above. In the first case, a column is removed from the beginning of the analysis and the vertical load is applied slowly, by using critical damping. The results shown in Fig. 15, in terms of plastic strain and M-N history of beams and columns, are in great contrast with those obtained during the transient analysis (Fig. 11-below and Fig. 14). In the pseudo-static analysis only the horizontal beam (element 71) reaches the failure surface, whereas the columns (elements 201 and 281) are in the elastic realm. The damage is limited to the first bay of the structure. It should be noted that in the comparison of the "static removal" versus the "dynamic" approach, the damage in the former one is also underestimated because the direct blast damage to the front façade is not given in the depicted results. Nevertheless, the induced damage is clearly underestimated in the "pseudo-static approach. This comparison reveals that in order to model progressive collapse due to blast loading, two aspects must be well described:

- The load redistribution, due to the sudden removal of a column, see also (Izzudin *et al.* 2008);
- The blast impact on the structure.

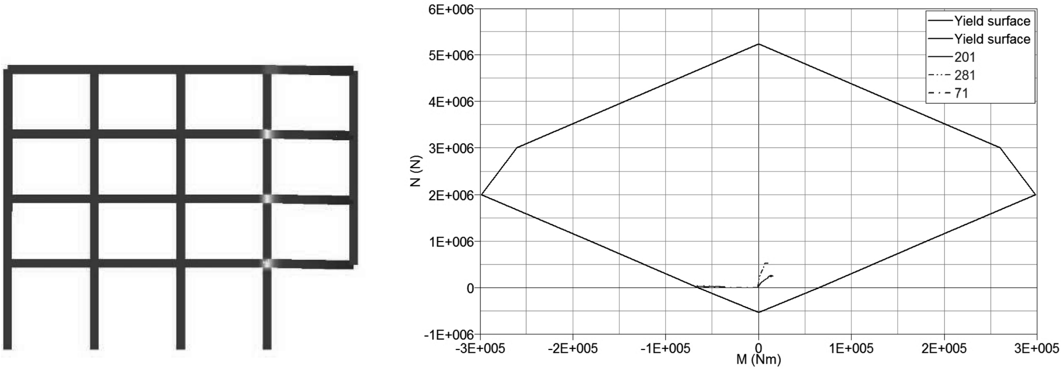


Fig. 15 Pseudo-static analysis for a 1000 kg TNT charge at 35 m standoff distance. (left) Equivalent plastic strain plot; (right) M-N trajectory and yield surface

This analysis also shows the necessity to consider the dynamic response of the whole building. When damage occurs and the residual strength of an element is reduced, it is stated that the overall response and damage of the total structure cannot be derived from the sum of the individual elements. The total structure has to be considered. Consequently, engineering methods like the SDOF-method can help to identify whether severe damage of individual elements would occur, but are not suitable to analyse the building response.

3.4 3D RC Beam-shell model

This section aims at illustrating the differences between the 2D model, above studied, and a 3D model. Floors are modelled as 0.15 m thick shells, and beams and columns are modelled as in the 2D model discussed above. The reflected blast pressure is computed automatically by means of CONWEP (*LOAD_BLAST), which is featured by LS-DYNA. This accounts for the correct spatial pressure distribution including clearing effects, unlike the 2D model. Referring to the comment in section 3.1, also for the 3D-situation the influence of (multiple) reflections in a real urban environment are not considered in this paper because of the focus on the structural response aspect. To avoid floor interpenetration during an eventual progressive collapse, self-contact is used in the analyses. Shell elements are used on the façade to transfer the blast pressure to the structure. By default, it is assumed that the façade elements are massless and have no strength.

Fig. 16 and Fig. 17 show plastic strains of the beam and shell elements respectively. Fig. 18 shows the floor plots of uniaxial concrete strain, which is a good indicator of the amount of damage by crushing. For the 100 kg charge, plastic deformation is limited to the front of the building. Only the corner columns and the top beams are affected. Damage is caused by bending. There is only some damage on the top floors, next to the façade. For the 500 kg charge, there is a considerable amount of plastic deformation in the beams and columns at the front face of the building, which extends up to the second row of columns, as well as to most of the building joints, and the foundations. The floors between the first and second rows of columns are rather damaged. Despite of the amount of the plastic deformation, the building does not collapse. However, the displacements of the top floor are inadmissible. The total amount of damage is so large, that probably it cannot be repaired. For the 1000 kg charge, the amount of damage is even larger and

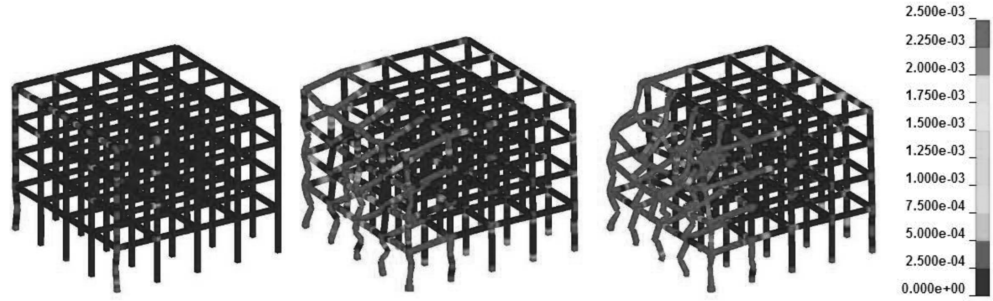


Fig. 16 Beams and columns: equivalent plastic strain plots for different blast charges, stand-off distance = 35 m: (above-left) 100 kg TNT; (above-right) 500 kg TNT; (below) 1000 kg TNT

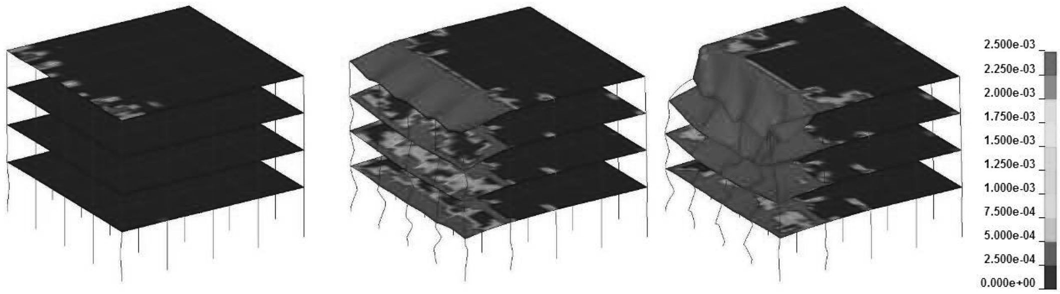


Fig. 17 Floors: equivalent plastic strain plots for different blast charges, stand-off distance=35 m: (above-left) 100 kg TNT; (above-right) 500 kg TNT; (below) 1000 kg TNT

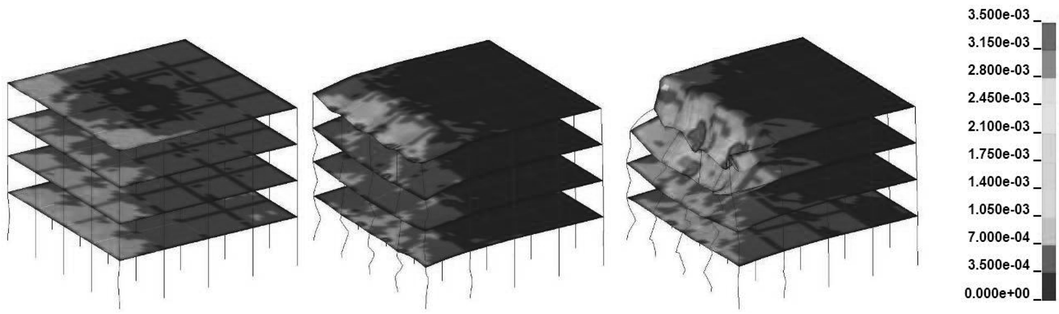


Fig. 18 Floors: uniaxial concrete strain plots for different blast charges, stand-off distance=35 m: (above-left) 100 kg TNT; (above-right) 500 kg TNT; (below) 1000 kg TNT

more widespread than before. The front part of the building fails, in a progressive collapse fashion.

Comparing the 2D model with the mid frame of the 3D mode, one can see that the amount of damage predicted by the 3D model is larger than the damage predicted by the 2D model. Obviously, to determine the extent of damage of a building after a blast explosion, a 3D analysis must be done. Since progressive collapse is closely linked to the extent of damage, 3D analyses are preferred. It should be noted, that the results of the presented 2D and 3D models are not fairly comparable, because the blast pressure in each case is not the same. In the 2D model the blast pressure is uniform and acts only on the front of the façade. In the 3D model the blast pressure acts on the four facades and the roof, and varies spatially as computed by CONWEP.

3.5 2D Solid-rebar model

Finally, we have studied a 2D frame of the building using a more advanced approach that was also applied in the analysis of the RC-plates in section 2. Concrete is described by means of solid elements and the K&C concrete material (*MAT_72R3). Rebars are described using beam elements and a strain rate dependent von-Mises plasticity material model (*MAT_PLASTIC_KINEMATIC). Perfect bonding between concrete and rebars is assumed. Due to symmetry in the out of plane direction, only one half of the structure is modelled. The mesh is made of approximately 23.000 solid elements and 8500 beams. The same three load charges are studied, 100 kg, 500 kg and 1000 kg of TNT, at 35 m standoff distance. A uniform full reflected blast pressure acts on the façade, as given in Table 1. This makes a direct comparison with the 2D model of section 2 possible. The computing time was 2 hours, using the same platform as with the beam model, which took 2 minutes. The meshing time was also much longer, mainly because every rebar had to be individually modelled.

The plots of Fig. 19 show the amount of damage for the three load cases. These plots bear a close resemblance with those shown in Fig. 11.

In Fig. 20 the displacements of the beam and solid-beam models have been compared, namely the midspan deflection of the first floor, adjacent to the façade (between columns 321 and 361). During the preload phase, there is a good agreement between the two models. Yet, after the blast, the discrepancies between the two models are significant. The beam model predicts higher deflections, which reveals a lower stiffness (more damage). It is worth to point out, that although the elastic and strength parameters of the two models are similar, default values have been used for the softening part, which are different in each case and contribute to the observed discrepancies. Another aspect is the rate dependency, which the beam RC concrete model (*MAT_172) lacks. The calculated stress rates are different for both methods. High frequency phenomena related to the induced stress wave by the explosion can be captured by the solid elements, but these are averaged to structural response in the beam and shell elements. Consequently, the solid modelling requires a more advanced, more accurate concrete material model than the beam model, because it has to represent also the concrete behaviour for the high loading rate regime. As also mentioned in section 2, the knowledge on the rate dependency of concrete at high loading rates is still limited, especially for the softening behaviour, and not yet properly covered in most numerical material models. Therefore, the beam element model and the solid element model may lead to different response and damage prediction.

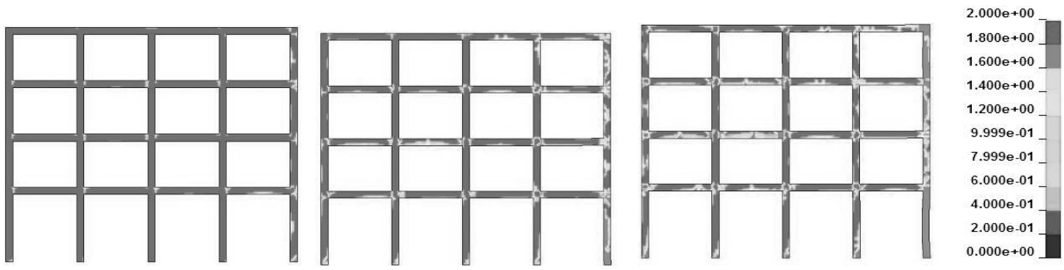


Fig. 19 Damage plots for different blast charges (stand-off distance = 35 m): (above-left) 100 kg TNT; (above-right) 500 kg TNT; (below) 1000 kg TNT

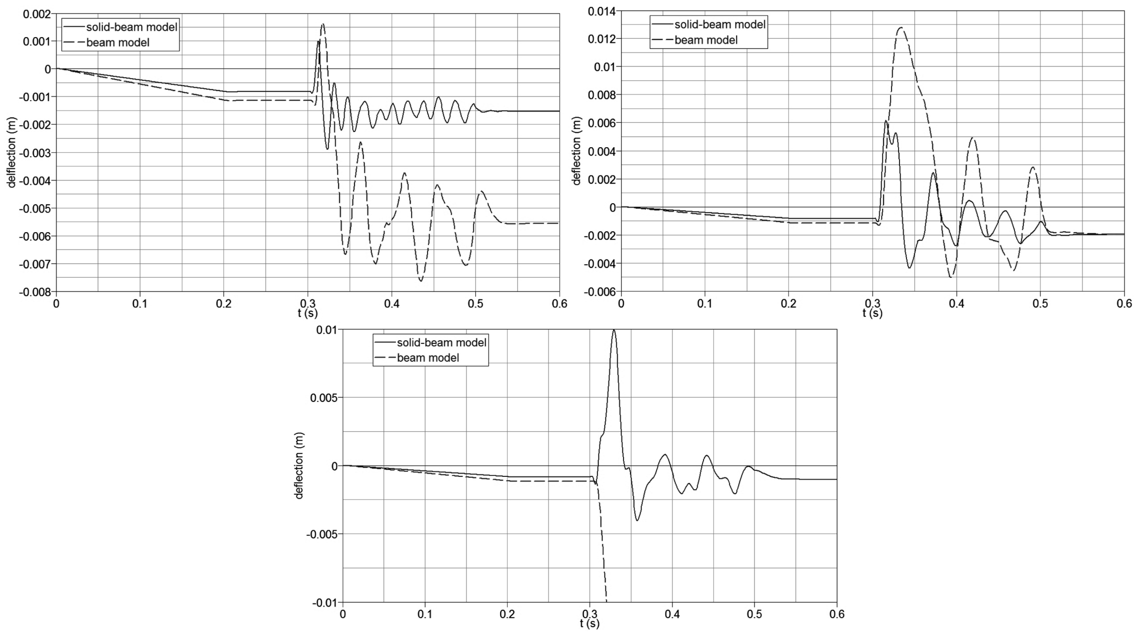


Fig. 20 Deflection history for the solid/beam (blue) and the beam smeared model (red). Stand-off distance = 35 m. (above-left) 100 kg TNT; (above-right) 500 kg TNT; (below) 1000 kg TNT

For more information on the rate dependency of concrete, the authors refer to (Weerheijm *et al.* 2007a, 2007c, Vegt *et al.* 2008).

3.6 Concluding remarks concerning the case study

The results of the computational analysis in the case study are:

- Progressive collapse is a highly dynamic process. Blast load, inertia effects, and strain-rate behaviour must be well described.
- The blast load has been modelled either assuming a uniform pressure distribution (2D model), or with CONWEP (3D model). Obviously, the former is less accurate than the latter. To account for (multiple) reflections occurring in real urban situations, engineering methods and more advanced techniques are available: CFD (computational fluid dynamics) or ALE (arbitrary lagrangian eulerian).
- Progressive collapse is often modelled in literature by studying the dynamic response of a building, upon the sudden removal of a column. This approach is only valid for small localised explosive loads, since it neglects the blast impact on the structure and consequently the internal wave propagation and dynamic forces in the structural elements.
- Inertia effects have been taken into account by performing an explicit dynamic analysis (LS-DYNA) for the entire loading history, including the preload stage. A more efficient approach is to perform an implicit dynamic analysis during the preload (dead load) stage, and switch to an explicit dynamic during and after the blast explosion.
- Strain rate hardening is accounted for in the more advanced concrete material model (*MAT_72R3). For the smeared concrete model (*MAT_172), the strain rate effect has been

approximated by using a constant dynamic increase factor. The consequences of not taking into account the strong rate dependency of concrete in the high loading rate regime should be studied in detail in order to improve damage prediction and enable the comparison of solid- versus beam/shell element modelling.
- The applied concrete models feature softening, which allow the description of progressive collapse, without needing an ad-hoc failure criterion.
- The main advantage of the simpler smeared model is that the pre-processing (meshing) time and the computing time is short. This allows modelling large structures, even in 3D, in a very short time. This smeared approach can be used for a first estimate of progressive collapse. On a second step, the more advanced model is recommended.

Based on these findings, it is stated that the induced explosion damage to a RC building can be quantified accurately enough using the beam-shell element modelling in a 3D-format, to generate the required input for the resilience assessment tool.

4. Structural damage and loss of functionality

The conclusion from the research presented in the previous sections was that the damage to a building and the individual structural elements can be quantified using beam and shell element modelling. The next step in the research programme on the resilience of urban infrastructure is how to translate this information to a damage parameter that can be related to the function of e.g., a room, an alley or an elevator? In this section the proposed procedure is introduced.

The building is divided into volumes or cells in between structural elements (beams, columns and plates). The damage, stress and strain states of the structural elements are known from the FE calculations. Various options have been examined to use and translate the code-output objectively to a "volume damage" parameter. The selected approach is the following. Additional volume elements are placed in each cell, with negligible mass and stiffness, so they do not affect the structural response. From the shear strain of the volume elements the damage is obtained. These volume elements can be used as "damage gauges" upon calibration.

To calibrate the damage, a numerical experiment is performed. A cell, which consists of structural elements and volume elements, is statically loaded in shear up to failure. The basic assumption is that the dynamic failure mode is similar to the static one. The cell fails when instability occurs, indicated by a steep increase of the kinetic energy and a plateau in the internal energy, $U_{max} = 1.2e5$ Joules. A this point, the corresponding deformation, $\varepsilon_{max} = 0.13$, is identified as failure and corresponds to a damage-gauge value of 1. This procedure is illustrated in Fig. 21.

This procedure has been applied to the 4 storey RC building of the case study. The additional effort in pre-processing by adding the volume elements as well as the additional cpu-time is negligible. Fig. 22 shows the calculated structural deformation for the 4-storey building due to the 100, 500 and 1000 kg explosion, and the corresponding damage levels.

This procedure has been applied to the 4 storey RC building of the case study. The additional effort in pre-processing by adding the volume elements as well as the additional cpu-time is negligible. Fig. 22 shows the calculated structural deformation for the 4-storey building due to the 100, 500 and 1000 kg explosion, and the corresponding damage levels.

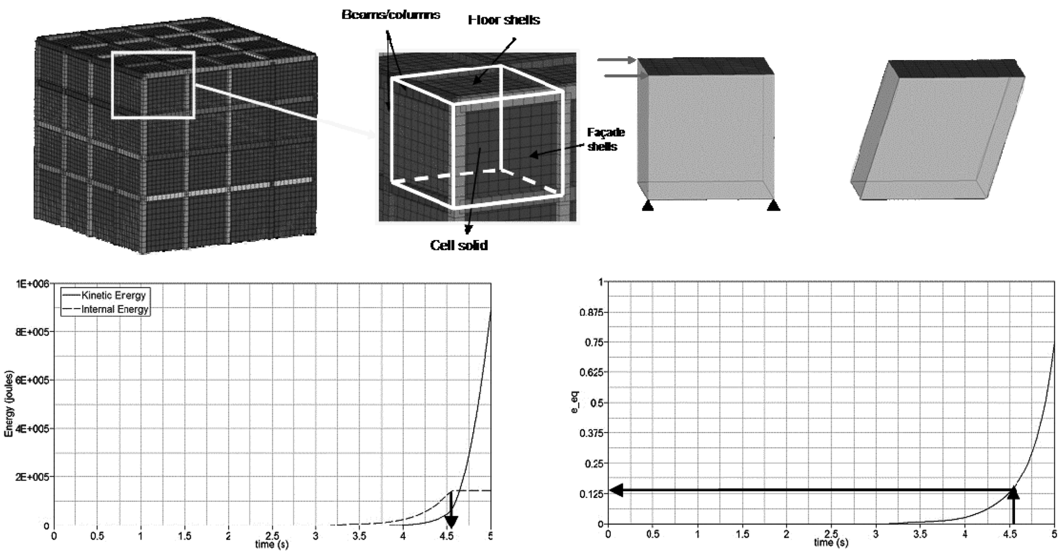


Fig. 21 Damage calibration procedure. (top left) a cell, consisting of structural and a volume ('gauge') element; (top-right) shear calibration test; (bottom-left) kinetic and internal energy history, $U_{\max} = 1.2$ MJoules; (bottom-right) shear deformation history, $\varepsilon_{\max} = 0.13$

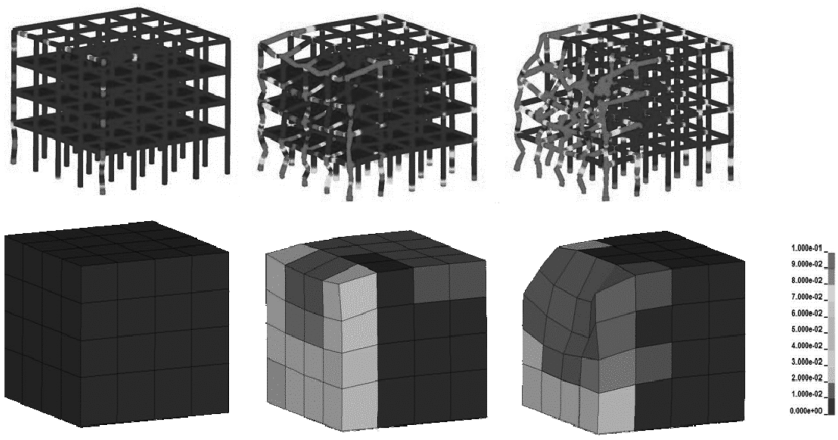


Fig. 22 Illustration of the relation between the structural damage and the damage indicator for the three different blast charges: (left) 100 kg, (middle) 500 kg, (right) 1000 kg of TNT

Following this procedure, the FE-calculation gives results for the structural response, the induced structural damage and provides *damage values* for all the predefined volumes. The remaining action is to define relations between functionality loss and damage values. This falls beyond the scope of the current paper.

5. Conclusions

The development of an assessment method for the resilience of a city confronted with a terrorist bomb attack is the background of the research reported in this paper. The possibilities to quantify the damage to RC buildings due to external explosions have been studied, starting from single structural elements, to the whole structure. Shock tube tests have been performed and analysed using the SDOF-engineering method and advanced FE-simulations. Detailed conclusions are given in the sections 2.5 and 3.6.

The main results and conclusions are:

- To quantify the structural damage to RC buildings due to external blast, the whole structure has to be considered. The interaction between the structural elements determines the dynamic response of the whole (bearing) system and therefore the final damage.
- Finite element methods are suitable to study and quantify damage and the structure's dynamic response. Engineering methods (single or multiple degree of freedom systems) fail because the true dynamic response (higher eigenmodes, non-linearities, interaction with the rest of structure, etc.) can not be captured.
- Using a nonlinear explicit finite element code, the explosion damage and the residual bearing of capacity of multi-storey RC buildings has been quantified. Nevertheless, a combination of implicit-explicit analysis is more adequate for this type of analysis.
- A method is proposed to relate structural damage to volume damage. It remains to link the latter to the loss of functionality of buildings.
- The developed methods and gained knowledge form the basis of the resilience assessment tool for city infrastructure which is also being developed at TNO Defence Security and Safety.

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