Design concept for high strength composite members subjected to exceptional loads

M. Gündel[1] & B. Hoffmeister[1]
[1]*RWTH Aachen University, Institute for steel structures, Aachen, Germany*

ABSTRACT: Recent developments of the global political situation have led to new challenges for the design of buildings for exceptional loads. The design of buildings subjected to impact and blast loads is normally related to heavy concrete structures. Nevertheless, for general buildings two different engineering concepts are already used: the Key element method and the Alternate load path method. For ductile steel-composite structures, resisting dynamic loads more effectively through plastic deformations, a novel method based on residual strength of damaged key elements is introduced. Those key elements are designed to resist the dynamic short term actions utilizing plastic deformations. In a second step the global structural stability is checked considering the residual strength of the deformed key element. Partial load redistribution may be considered. The Residual strength method guides a way towards a more realistic and hence economic design.

1 INTRODUCTION

Nowadays there is also in Germany a new challenge for structural engineers to consider exceptional loads like blast and impact not only for military and nuclear facilities but also for civil structures like office buildings, embassies, further governmental as well as public buildings. Recent developments of the global political situation combined with increased potential of terroristic attacks have led to an increased demand for enhanced safety of buildings and for special passive protection measures. This is also originated by a change in the nature of terroristic acts, as instead of attacking single persons the new aim of terrorists is to affect a maximized amount of people. Similar scenarios exist also for industrial structures, in particular within industrial facilities with increased risk of accidents and possible serious consequences for the population and environment. The main differences from military facilities and nuclear power plants to the new aforementioned application fields are the type of structure and the accessibility. The first buildings are heavy reinforced concrete structures usually in the open countryside while latter are often characterized by a light open architecture within populous areas. Typical hazard scenarios include the impact of a vehicle as well as blast waves induced by an explosion followed by fire. The design of structures that may be exposed to such actions primarily aims at the protection of human lives and of building equipments or other goods. On the other hand, damage to particular structural regions or members should be limited in order to achieve a sufficient degree of robustness against full or partial progressive collapse of the building.

2 DESIGN CONCEPTS AGAINST EXCEPTIONAL LOADS

2.1 *Existing design concepts*

Heavy reinforced concrete structures subjected to exceptional loads can retain their full integrity, if they are designed similar to shelters. However, this is not possible for structures with linear structural elements, e.g. made of steel or composite, and is not aspired for public buildings with an outstanding architecture. In these cases the aim is to limit the structural damage and prevent the building for progressive collapse. Currently three main concepts are pursued with regard to the design of robust structures: (i) the key element method, (ii) the alternate load path method and (iii) the isolation of collapsed regions.

The first method is based on the idea of providing structural "key elements" which in case of an exceptional load must survive in a largely undamaged state whereas other structural elements are allowed to be partially or fully damaged (Figure 1a). The design of key elements is usually performed by verifying their ultimate resistance against an equivalent static load. This method is applicable to heavy structural elements, where the inertia of the heavy mass is sufficient in order to allow for neglecting dynamic effects. In several cases this approach leads to space and material consuming (and thus expensive) solutions and is not appropriate for open architectural solutions. The shortcoming of this method is that the ductility of steel and steel-concrete-composite members which are capable of resisting severe dynamic loads by means of energy dissipation due to plastic deformations is not considered.

The second concept, involving the provision of alternate load paths, assumes failure of particular structural elements under the exceptional load and aims at the redistribution of gravity loads initially supported by the damaged members among adjacent undamaged structural parts (Figure 1b). This approach usually aims at the activation of membrane behaviour of horizontal members (e.g. beams and floors) and of tensile forces in the columns above the damaged one. The commonly used design procedure starts with the removal of one element followed by a static verification of the remaining structure. Depending on specific scenarios, additional members are removed subsequently. This method usually does not consider the dynamic effects appropriately and also neglects the presence of residual resistance in partially damaged members. Moreover, the calculation of the alternate load path is based on the assumption that the damage is limited to the removed member(s). In reality, it is to be expected that – particularly in case of an explosion – several structural members suffer from the load. Furthermore, the verification of the alternate load path does not consider the response of the structure (during and after the action) which may be crucial for local and global stability. The method may be suitable, with extensive calibration and validation, for regular buildings of moderate height. Nevertheless, the applicability of this method to high rise buildings, industrial facilities and demanding architectural solutions is very limited. In these structural configurations, gravity loads are often borne by few highly loaded members – in such cases the provision of measures for redistribution of loads is either not possible or at least very expensive. Furthermore, with this approach it is not possible to design safety zones within buildings.

The third concept – prevention of the progressive collapse of structures by isolation of the affected region - has been used until now primarily for bridges (Starossek 2005). The structure is designed and considered to consist of several independent subsystems, separated e.g. by means of hinges, to limit the extension of failure under an exceptional load to a pre-defined region (Figure 1c). By applying this method a local failure does not lead to a disproportional collapse of the structure. However, this method can only be used for buildings with a limited number of stories in order to keep the extent of the collapse zone within controlled, predictable and acceptable limits. This method is also inappropriate for the development of safety zones.

2.2 *Starting point for a new design concept*

The current state of the art has led to a considerable scepticism towards the practicable application of steel and steel-concrete-composite solutions for blast and impact resistant structures. The main reasons are not the mechanical properties of these materials but the missing availability of appropriate and applicable design concepts. Lessons from catastrophic events in the past show an excellent resistance and remarkable residual strength of steel structures subjected to accidental actions. For example, the structure of Exchequer Court in St. Mary's Axe (London, 1992) was heavily damaged by the explosion of a truck bomb. Some steel columns in the ground floor were plastic deformed up to L/10 without collapse of the building. In another case, the pre-heater tower of a cement plant

(Philippine, 2007) was severely damaged by the collapse of a neighbouring reinforced concrete silo. The corner column with a length of 23 m suffered 700 mm plastics deflection (L/30) and has still provided a residual axial capacity of about 20 %, which preserved the global stability of the structure until safeguarding and rehabilitation measures could be applied.
In the following an alternative method is described, which enhance the security of buildings by measures utilizing available capability of materials and structural parts in combination with an appropriate design concept. It is based on the exploitation of the residual strength of damaged members and takes advantage of the ductile behaviour of steel and composite members (Residual strength method, Figure 1d). The concept integrates experience and knowledge of seismic engineering as performance based design, ductility requirements, capacity design, etc. The main differences with seismic scenarios are related to the time dependence of the actions and global and local dynamic response of the structure. The method leads to robust and redundant structures without interfering with the intended functions of the structures and their aesthetic appearance. It enhances the resistance of buildings against progressive collapse and provides tools to design "safety zones" within the building for extraordinary sensitive subjects. Furthermore, this is available at a minimum of additional costs. The new concept bridges the gap between the Key element method and the Alternate load path method.
The concept based on the results and experience gained by a collaborative European RFCS-project under participation of ArcelorMittal, HOCHTIEF Construction AG, Imperial College London, Universität Karlsruhe und RWTH Aachen University (COSIMB 2008). The work involved static as well as impact and blast tests on partially encased composite columns and composite column-wall systems. These are reinforced concrete walls attached on a steel or composite column acting as composite beam under horizontal loads and offer the design of "safety zones".

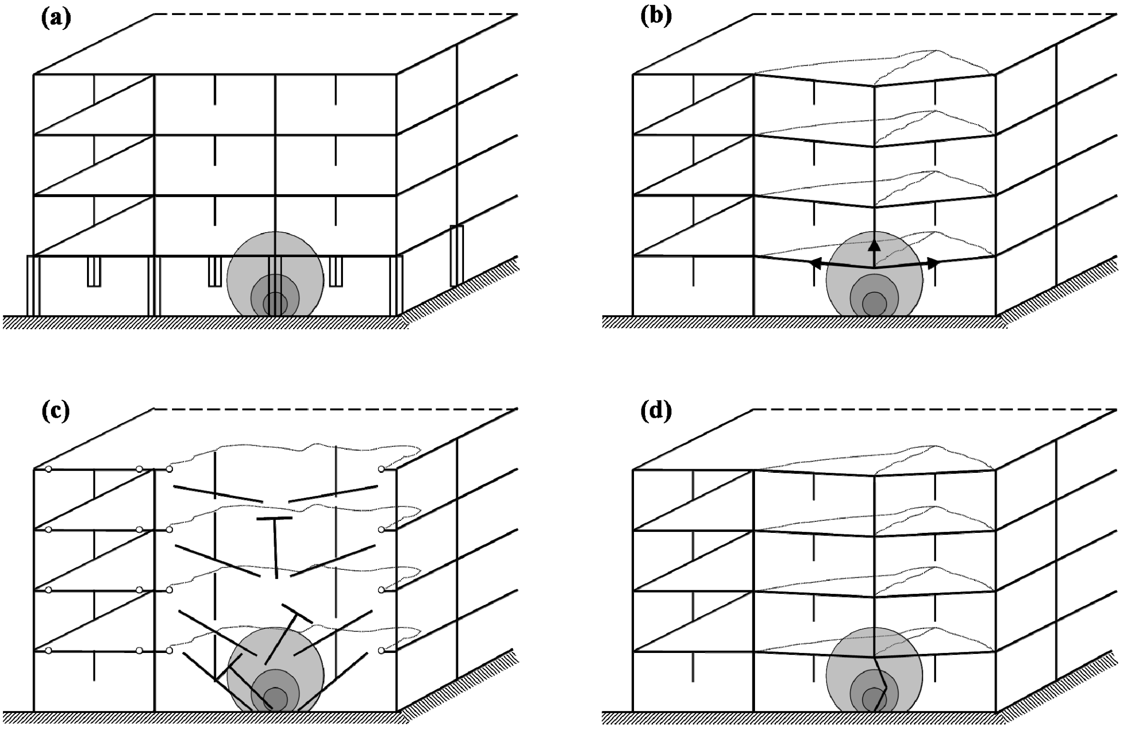

Figure 1. Strategies for robust structures: a) Key element method, b) Alternate load path method, c) Isolation of collapsed region, d) Residual strength method.

3 RESIDUAL STRENGTH METHOD

3.1 *General*

Results of the experimental and numerical investigations on composite members subjected to exceptional loads have revealed an excellent ductile behaviour and a considerable residual capacity of the tested specimens (COSIMB 2008). The impact and blast load applied to these members caused significant plastic deformations, which however are not to be seen as equivalent to a failure of the member. The residual capacity depends on the fulfilment of basic requirements concerning the maintenance of the structural integrity of the member itself as well as in relation to the global structure. These requirements pertain in particular to the connections which have to be designed with sufficient overstrength in order to prevent a separation of the member from the structure. The members potentially affected by the exceptional loads are designed under consideration of energy dissipation by means of plastic deformations. The verification of the global stability of the structure after the exceptional loads event is performed in the following, taking into account possible dynamic response and the residual strength of the deformed members.

3.2 *Dynamic loads*

Due to their particular motivation, terroristic attacks are difficult to assess by means of mathematical models. However, contrary to design concepts based on unspecific actions, e.g. the Alternate load path method, the identification of specific loads is essential for the Residual strength method. A pragmatic and particularly suitable procedure for man-made hazards is shown in the FEMA-report 426 (FEMA 2003).
As blast and impact are extreme transient load cases, they are significantly different from static ones with the same maximum force amplitude. The main load parameters of a vehicle or plane impact are the velocity, mass and contact stiffness of the impact body and the impact area. Typical masses and contact stiffness for different vehicles are summarized in (COSIMB 2008). The velocity of the impact vehicle should be estimated under consideration of the site conditions.
The major medium for energy transfer released by an explosion is the shock wave. It is characterized by the peak pressure, the duration and the impulse, which is the area under the pressure-time curve. As the duration of the shock wave is very short, the maximum response of the structure is reached with a significant delay due to the inertia of the member. Several methods are available for determining the blast load parameters, ranging from simple nomograms through empirical expressions and computational tools. For design procedures presented hereafter a simplified triangular load with equal peak pressure and equivalent impulse achieved by adjustment of duration of the blast wave can be applied.

3.3 *Deformation behaviour and limit states*

Advanced dynamic calculation methods need the realistic resistance-deflection curve and the deformation limits of the structure. As under high dynamic loads the response of the structure is initially limited to the affected element, a helpful simplification can be done by analyzing the single member independently from the global structure (Figure 2). Non-linear transient finite element simulations of impact and blast loads on structural members as part of a complete 3-dimensional structure were carried out. They show a relative small influence of the surrounding structure on the dynamic behaviour of the single member during the transient loading (Figure 3). The maximum deflections are similar for the structural member as part of a global structure (1) as well as for a single column simple supported with (2) and without (3) a mass of 5 t at column head. A single column with axial restraint support conditions (4) underestimate and a column with an axial force of 500 kN (5) overestimates the maximum deflection. Hence, the maximum deflection can be (conservatively) determined on a single member simple supported. However, special attention needs to be paid to the detailing of the connections to bear horizontal as well as axial support forces.

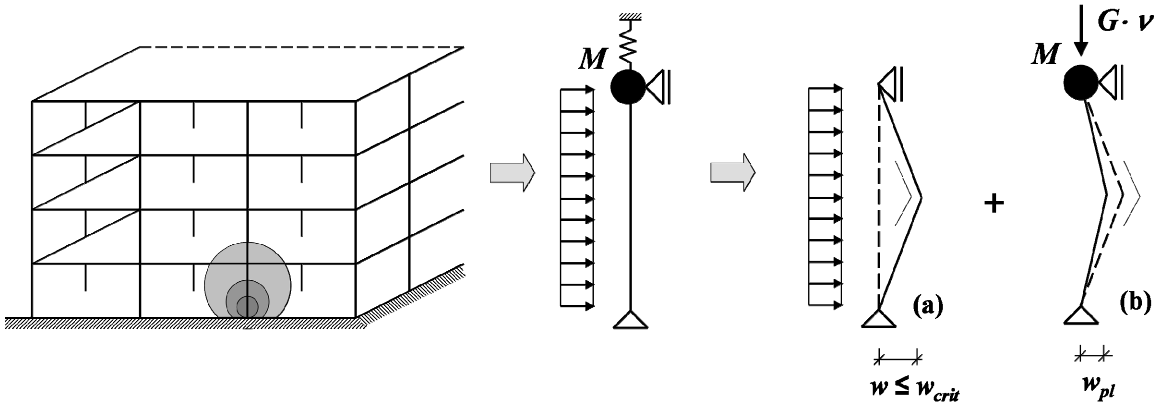

Figure 2. Design of structural member during (a) and after exceptional loading (b).

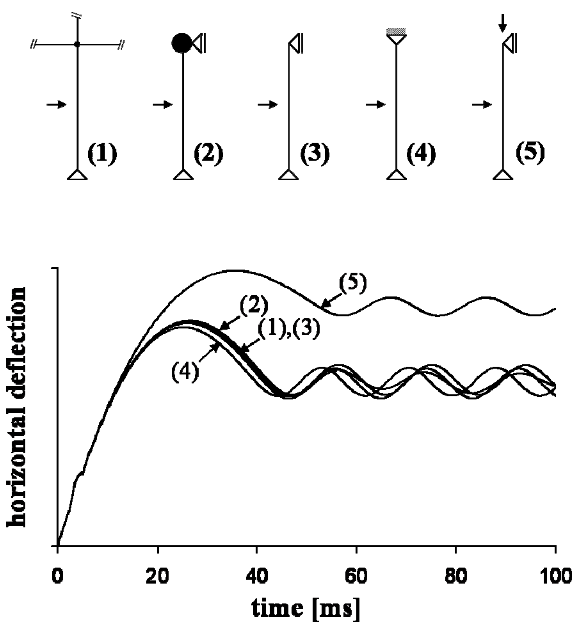

Figure 3. Influence of boundary conditions on the response.

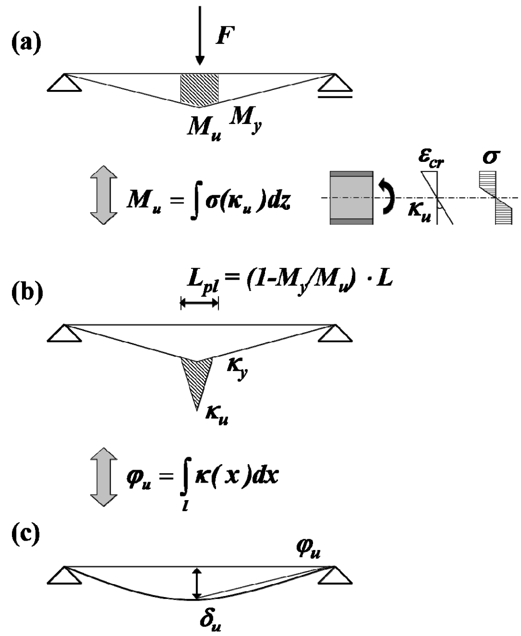

Figure 4. Analytical determination of the ultimate deflection.

Furthermore the numerical simulations show that the resistance-deflection behaviour under dynamic loadings is affine to the curve under a corresponding static load. The only adjustment incorporated is an enhancement of the ultimate resistance due to strain rate effects, which leads to an increase of the steel yield strength and the concrete compressive strength (ca. 20%). Resistance-deflection curves can be obtained experimentally, by numerical simulations and approximately by analytical calculations.

For steel and composite columns a bi-linear approach is normally sufficient, which is characterized by the initial stiffness and the plastic capacity of the member. Static tests on partially-encased composite columns under bending in the major axis direction show a considerable higher ultimate moment capacity than obtained from section analysis based on plastic distribution of stresses M_{pl} (Elghazouli 2008). This can be ascribed to strain hardening in steel but also to confinement effects in concrete. Latter can be considered by the use of a concrete strength enhancement factor of 1.5-2 in effectively confined areas. As local buckling of the flange leads to a sudden release of concrete confinement accompanied by a gradual reduction in moment capacity, the critical buckling strain can be used as a reasonable deflection limit. For partially encased composite columns combining fundamental buckling theories with experimental results yields to a simple relationship, where the critical buckling strain can be predicted as a function of the commonly-used plate slenderness λ_p (Elghazouli 2008, COSIMB 2008):

$$\lambda_p = 1 \frac{1}{\sqrt{1 + 0.5(\frac{\varepsilon_{cr}}{\varepsilon_y} - 1)}} \tag{1}$$

where ε_{cr} = critical buckling strain; and ε_y = yield strain.

Based on the strain limit definitions the deflection limit can be estimated by the procedure described in Figure 4: (a) The curvature κ over the length of the member is determined related to the moment distribution at the limit state by integration of the corresponding stress distribution over the height of the section. The ultimate curvature is defined by reaching the critical strain ε_{cr} in the flange under compression. (b) Integration the curvature over length of the member leads to the rotation φ_u of the member. This requires an evaluation of the plastic hinge length which is estimated as $L_{pl} = (1-M_y/M_u) \cdot L$, where L is the length of the member. A conservative approximation is a linear distribution of the curvature within the plastic hinge zone. (c) The ultimate deflection δ_u of the member is obtained by integrating the rotation. Typical deflection limits for composite columns under major axis bending for Class 1 sections exceeds a ductility ratio $\mu = 10$, where μ is the ratio of total ultimate deflection to deflection at the elastic limit. For pure steel columns a value of $\mu = 20$ is suggested by (Mays 2005).

Similar considerations can also be carried out for composite column-wall systems, which consist of a RC-wall attached to a composite column acting as a composite section. For such sections a strain limit of ε_{cu} = 0.35 % can be applied for the unconfined concrete chord. However, even if the load level decreases after crushing of the concrete slab, there is still a remarkable residual moment capacity. Hence, it is reasonable to establish a second resistance level, which can be gained by reducing the concrete strength by $f_c' = f_c \cdot \beta$ with $\beta = 0.2$ in the plastic section capacity. A sufficient correlation with test results could be obtained by estimating the plastic hinge length with two-times the height of the cross section (Hauke 2008).

3.4 *Dynamic structural response under impact loads*

As mentioned above the determination of the structural response under high dynamic loads can be reasonable simplified, if the behaviour is analyzed on a separated member. However, afterwards the time-shifted interaction between member and global structure has to be considered. To investigate the dynamic response of composite members 6 impact tests were carried out (COSIMB 2008). In the experimental investigations composite columns and composite wall systems in steel grade S460 and with a concrete strength of C40/50 were tested as single span girder, simply supported.
In the tests a vehicle with a mass of about 1.4 t and contact stiffness similar to real cars crushed into the specimens with initial velocities between 6.0 and 14.5 m/s. The impact caused a clear plastic hinge formation in the affected area of the composite columns. The impact load history - calculated by multiplication the impact mass with its acceleration - oscillates around the plastic moment capacity of the member. The concrete slab of the composite wall system failed after some Milliseconds and the load histories show a second load level in the range of the residual strength of the static tests. Therefore the dynamic test results confirm the findings made in the analytical calculation to use an affine static load-deflection curve in the dynamic analysis. Even with extensive plastic deformation the integrity of the member is guaranteed and even a remarkable residual strength can be achieved.
In order to obtain a better understanding of the dynamic behaviour, transient finite element analyses of the specimens in the tests were carried out. The impact scenario were simulated with three different finite element models with idealised mechanical properties (Figure 5 and 6): (1) boundary conditions and properties of the member corresponds to the performed tests; (2) boundary conditions corresponds to the tests, but the mass of the member is set to zero; (3) equal to model 1 but without supports - the specimen is able to move free after the impact. After 8 ms model 1 and 2 show coincident impact forces equal to the member moment capacity. However, there is a vibration of the time-impact force curve of model 1 as the distributed mass over the member accelerates, which cannot occur if the mass is missing (model 2). Within the first 8 ms the behaviour of model 1 and 2 is essential different. Due to the inertia of the member in model 1 the impact force rises suddenly to a peak value, while the load in model 2 increase rather linear till the maximum impact force is achieved. Although the structural member in model 3 is not supported and can move freely the behaviour is equal to model 1 in the first 2.5 ms.

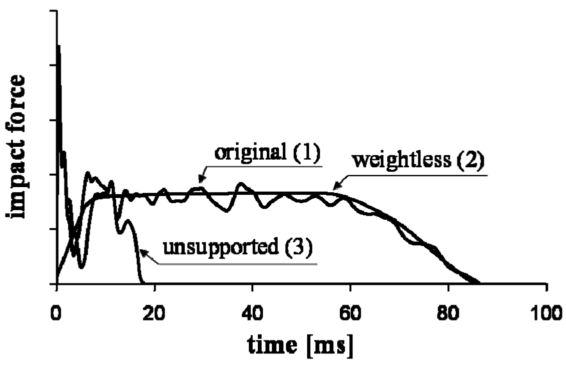

Figure 5. Impact force history for finite element models.

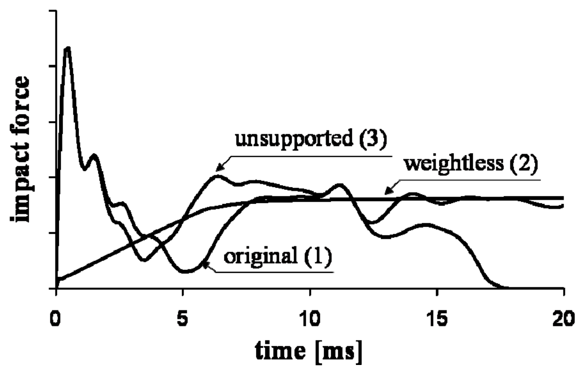

Figure 6. Impact force history for finite element models (close up).

For simplicity the duration of an impact can be subdivided in three phases: (i) the impact body crashes into the member until both bodies have the same velocity, (ii) both bodies move together and the member become deformed and (iii) the impact body is rebounded in the opposite direction due to the accumulated elastic energy. The behaviour in phase 1 can be described by means of principles of a plastic impact. The common velocity of both bodies after the impact can be determined by equation (1):

$$\bar{v} = \frac{M_i v_i}{M_i + M_s} \tag{2}$$

where M_i = mass of vehicle; M_s = mass of structural member; and v_i = initial velocity of impact body.
In phase 2 the maximum and plastic deformation of the structural member are determined by the energy balance approach:

$$\frac{1}{2}(M_i + M_s)\bar{v}^2 = \frac{1}{2}F_{pl}w_{el} + F_{pl}w_{pl} \tag{3}$$

where F_{pl} = plastic resistance; w_{el} = elastic deflection part; and w_{pl} = plastic deflection part.
If in addition to the maximum deformations time dependent internal forces are required, e.g. in order to verify the shear capacity of the member and to design the connections, a generalized mass-spring-model provides reasonable results. For the impact scenario the model has two degrees of freedom (TDOF) and consists of two non-linear springs representing the structural stiffness of the member and the contact stiffness, two masses representing the effective mass of the structure as well as the mass of the impact body and two dashpots with effective damping coefficients. The mass, damping and resistance quantities in the model are not directly equal to that ones in the real structure. They have to be converted by a shape function that approximates the fundamental mode of vibration of the member (Chopra 2001). The non-linear equation can be solved by means of the Newmark time integration scheme and the Newton-Raphson method. Joints should be carefully detailed, as the initial dynamic response of the member to the impact body may cause higher support forces in the moving direction of the impact body than in the opposite direction, when the ultimate plastic capacity is reached.

3.5 *Dynamic structural response under blast loads*

Besides the impact tests, blast tests were carried out on composite members with the same properties, dimensions and boundary conditions than in the impact tests. The tests were performed in a shock tube, which enables to apply pressure time functions similar to free field explosions with approximately 1.1 t to 6.5 t TNT in a distance of 19 m to 28 m but with fewer amounts of explosives. In the tests with composite columns only slightly plastic deformations were measured, as the affected surface of the column is relative small. The specimens of the composite wall systems were assembled in such a way that the concrete wall faced the explosive. Due to the wider surface of the wall the effective load was considerable higher and maximum deformations up to 77 mm could be measured. After the maximum deflection is reached, the stored elastic energy accelerated the specimen backwards. The pressure of the still acting but decreasing shock wave damped the backwards-movement of the specimen but is not able to increase the deflection further. Again, the failure mode is crushing of the concrete slab and a clear plastic hinge formation is observed at mid span.
As the duration of blast waves is in the range of the Eigen-period of columns, the load is neither impulsive nor quasi-static and dynamic effects have to be considered. If only the maximum deformations are required, existing design charts of elasto-plastic single degree of freedom (SDOF) systems lead efficiently to reasonable results (see e.g. in (Baker 1983)). To apply this design chart, e.g. for a triangular pressure-time function, following input parameters are required: the ratio between duration of blast wave and Eigen-period of the member, the ratio of the plastic resistance to the peak pressure and the deflection at the elastic limit. Depending on these parameters the maximum deflection of the member is obtained. However, if time dependent internal forces are needed, the application of generalized mass spring models are recommended also for blast-loaded structures. The methodology outlined in the section before can be used for a SDOF system by removing the impact body from this system.

3.6 *Verifications during and after the exceptional event*

As mentioned above, basic requirement for the residual strength of a member is to preserve its integrity during the dynamic loading. On the one hand shear failure and separation of the connections have to be avoided in order to enable the mobilization of energy dissipating bending behaviour of the member and on the other hand the deflection limit must not be exceeded. Here dynamic effects during the transient loading are crucial for the design and the detailing of the connections. The maximum deflection and deformation limits can be determined by the methods described before. Afterwards, the global stability after the exceptional event has to be verified. For that purpose especially two parameters have to be considered: (i) plastic deformation of the column, which invokes an additional bending moment under axial loading and (ii) dynamic effects resulting from a delayed response of the global system. The plastic deformations and resulting additional stress can be estimated on the separated member using aforementioned methods. Dynamic effects in the global system should be determined by means of an appropriate equivalent static system including the participating mass of the upper storeys at the column head (Figure 2b). Due to membrane effects in the column induced by the horizontal acting load the mass above is being accelerated downwards (the connection of the column head is in tension); subsequently – due to the accumulated elastic energy – the column snaps back and elongates in opposition to the movement direction of the mass above. As a consequence, the commonly-adopted dynamic load factor of 2.0 to be applied to the vertically acting forces can be significant exceeded.

4 CONCLUSION

In practice the design of structures against progressive collapse is usually simplified by investigation of the extreme cases by assuming failure of single member(s) (Alternate load path method) or by the design of key elements for a quasi-static load (Key element method). The advantage of these methods is the applicability of simple static analysis without dynamic investigations. However, this is rather uneconomic for frame constructions in steel and composite.

The fundamental difference of the new approach to the existing concepts against progressive collapse is the consideration of the dynamic interaction between members and the global structure in conjunction with the residual strength of the deformed member. Thus, this method is based on the realistic behaviour of member and global system – including dynamic effects – which enables improved prediction and prevention of failure modes under exceptional loads. Furthermore, beneficial material properties as ductility and energy dissipation are considered, so that the ultimate state can be determined more realistically and the residual capacity of the member can be considered for global stability. Thus, an economic design of steel and composite structures for exceptional loads is possible.

Information about materials and members are usually limited to the knowledge of ultimate resistance - often there is a lack of information about ultimate deformations and residual strength. In this paper methods to estimate these values for composite column and column- wall systems are presented. Based on further investigations on the load-deformation behaviour of structural elements, this method can also be applied to other ductile members, e.g. reinforced concrete columns.

REFERENCES

Baker, W. E. et al. (1983). Explosion Hazards and Evaluation, Elsevier Scientific Publishing Company.

Chopra A. K. (2001). Dynamics of Structures – Theory and Applications to Earthquake Engineering, 2nd Edition, Prentice Hall.

COSIMB (2008). Composite Column and Wall Systems for Impact and Blast Resistance (COSIMB), Final Report, RFCS Contract-Number RFS-CR-04047.

Elghazouli, A. & Treadway, J. (2008). Ductility of partially-encased composite members under combined loading conditions, Proceedings of the EUROSTEEL 2008, 3-5 September 2008, ECCS European Convention for Constructional Steelwork

FEMA (2003). Reference Manual to Mitigate Potential Terrorists Attacks Against Buildings, Risk Management Series, FEMA-Report 426.

Hauke, B.et al. (2008). Experimental study of composite column - wall systems subjected to combined loading conditions, Proceedings of the EUROSTEEL 2008, 3-5 September 2008, ECCS European Convention for Constructional Steelwork

Mays, G. C. & Smith, P. D. (1995). Blast Effects on Buildings, Thomas Telford Ltd.

Starossek, U. (2005). Progressiver Kollapse von Bauwerken. Beton- und Stahlbetonbau 4, 305-317.