CHALMERS

Concrete Structures Subjected to Blast Loading

Fracture due to dynamic response

JONAS EKSTRÖM

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Abstract

The interest for building safety with regards to highly dynamic events such as close by explosion has accelerated after the recent decades of terrorist attacks. The interest has expanded from civil defence shelters and pure military targets to also include civil buildings used for different civil functions. Due to the rather rapid shift of area of interest, the general engineering community lack much of the knowledge and tools with which to design and evaluate structures with respect to dynamic events with very fast transient and high magnitude of peak loads. In this thesis, two studies of concrete structures are presented. Both studies focused on describing concrete with a combined damage and plasticity model. The aim was to study how different numerical models perform during highly dynamic events. Two main studies are presented.

In the first study the response in a concrete wall subjected to shock wave blast, leading to spalling failure was investigated. This situation is important since spalled-off fragments in protective structures may cause severe injury to the persons or equipment it is supposed to protect. Previous research indicates that spalling occurs when and where the tensile strength of a strain-softening material like concrete is reached. By using a simple uni-axial numerical model, this study shows that spalling instead occurs when the cyclic response from a blast wave gradually increase the inelastic strains in the concrete. This means that spalling takes place after several loading cycles and not necessarily at the depth where tensile strength is firstly reached. Furthermore, the study shows that the cyclic response in the material model used for numerical simulation has a decisive influence on the position and extent of the resulting spalling crack.

In the second study the response of reinforced concrete structures subjected to blast loads was investigated. Numerical models are used to evaluate the numerical response of a simply supported reinforced concrete beam and a one-way supported slab with a combined damage and plasticity constitutive model for concrete, CDPM2. Previous research has shown that strain-rate de- pendent material parameters might be overestimated for higher strain rate. In this study, these features are evaluated for reinforced concrete structures where bending is the dominating response. The numerical analyses indicate that fracture energy during tensile fracture and how this value is chosen have larger effects on the deformations of the structure than whether or not the strain-rate dependency of the material properties are taken into account. It is also concluded that mesh size and modelling techniques may have a large impact on the resulting response of the structure in the numerical analysis.

Keywords: concrete, blast load, numerical modelling, spalling, stain-softening, wave propagation, transverse/reverse loading, dynamic response, cyclic crack propagation

to My Sources of Inspiration

Preface

The work on this thesis was carried out between August 2012 and July 2015 at the Department of Civil and Environmental Engineering, Division of Structural Engineering, Concrete Structures at Chalmers University of Technology. The work was performed within the research project entitled "Blast and fragment impact: Reinforced and concrete and fibre concrete structures". The research project is a continuation of earlier work on concrete structures subjected to severe dynamic loading conducted at Chalmers by Morgan Johansson, Joosef Leppänen and Ulrika Nyström. The project is financially sponsored by the Swedish Civil Contingencies Agency.

I would like to thank my supervisor and examiner, Associate Professor Mario Plos, for his support and guidance. I would also like to thank my assistant supervisors, Assistant Professor Rasmus Rempling, Adjuct Professor Morgan Johansson and Senior Lecturer Joosef Leppänen, for their invaluable input. Further, I would like to express my gratitude to MScEng Björn Ekengren, the Swedish Civil Contingencies Agency, for his persistent support and patience. I also want to thank Professor emeritus Kent Gylltoft, MScEng Rolf Dalenius, the Swedish Fortifications Agency, PhD Ulrika Nyström, VK Engineering and Professor Karin Lundgren for their valuable contributions to the work in the project group.

I want to thank all my colleagues who have contributed to my development as an engineer and as a researcher; especially Håkan Lantz and Carlos Gil Berrocal.

Finally, I thank my friends and family. I would not have reached this point without you.

Thesis

This thesis consists of an extended summary and the following appended papers:

Paper A	J. Ekström, R. Rempling, and M. Plos. *Spalling in Concrete Subjected to Shock Wave Blast*. Submitted to "Engineering Structures"
Paper B	J. Ekström, R. Rempling, M. Plos, and M. Johansson. *Finite Element Analyses of Concrete Structures Subjected to Blast Loads with a Damage-Plasticity based Material Model, CDPM2*. To be submitted for international journal

Author's contribution to jointly written papers

The appended papers were prepared in collaboration with the co-authors. In the following, the contribution of the author of this licentiate thesis to the appended papers is described.

Paper A Responsible for planning and writing the paper. Made numerical implementations and carried out numerical simulations.

Paper B Responsible for planning and writing the paper. Carried out the numerical simulations and the evaluation of the the results.

Other publications related to the thesis

In addition to the appended papers, the author of this thesis has also contributed to the following publications:

Ekström, J., Rempling, R., and Plos, M. (2014). "Influence of Strain Softening on Spalling of Concrete due to Blast Load". In: *Proceedings of the XXII Nordic Concrete Research Symposium*. Ed. by The Nordic Concrete Federation. Vol. 2/2014. 50. Reykjavik, Iceland: Norsk Betongforening, pp. 157–160.

Contents

Part I
Extended Summary

1 Introduction

1.1 Background

For decades the focus of civil defence has been on civil defence shelters to withstand the threat against warfare and to protect civilians. Latter years, the general focus has often shifted towards the protection of specific targets which host important functions to society. Direct attacks towards the civil population, such as terrorist attacks, also post a more common threat today than for a few years ago. Due to the rather rapid shift of area of interest, the general engineering community lack much of the knowledge and tools to design and evaluate structures with respect to dynamic events with very fast transient and high magnitude peak loads.

Most structural analysis approaches have been developed with different simplifications and approximations, because of limitations in knowledge and to restrict the extent of cases studied. When it comes to response of concrete structures due to highly dynamic events, the load due to an explosion and the general response of statically loaded concrete structures are relatively well-known. However, despite decades of research within the area, many phenomena involved for the dynamic response of reinforced concrete structures are not yet fully understood.

Numerical modelling of dynamic events has been more common in recent years and is to a large extent treated as an available complement to physical testing. Even so, the tools and techniques to evaluate structures with numerical models are in many respects unexplored. Since some phenomena are not fully understood, it becomes rather complicated to determine the needed properties of these numerical models.

1.2 Purpose, aim and objectives of research

The purpose of the research presented in this thesis is to study numerical approaches involving highly dynamic events towards reinforced concrete structures. In the long-run, the research intends to study how fibre concrete can be used in reinforced concrete structures to increase the ability of withstanding highly dynamic events, such as explosions within the civil community. In order to do so, understanding of the underlying phenomena during highly dynamic events when material fracture appears in concrete are prerequisite.

The aim of the research presented in this thesis is to study how dynamic responses of concrete structures subjected to highly intense blast loads can be modelled numerically. Both simplified models and more advanced numerical models were used to investigate some of the effects of different material properties of concrete and reinforced concrete structures. Furthermore, the aim was to relate material fractures in concrete to design methods that can increase the safety of concrete structures, both for new designs and to strengthen existing structures.

The objectives have been to evaluate numerical approaches based on a combined damage and plasticity model to describe the constitutive laws of concrete materials. Furthermore, to describe the influence of constitutive laws on the response of brittle materials, such as plain concrete, subjected to impulse loading with regards to spalling.

1.3 Scientific approach and methodology

The chosen scientific approach consists of literature reviews combined with theoretical modelling benchmarked with numerical models. The numerical models aim to provide a simplified approximation of real behaviours and, thus, support the knowledge creation of the underlying phenomena. Numerical models can be motivated by relatively low costs compared to experiments. They also provide the possibility of studying the course of event for any given sequence in time and to study responses which can be difficult to measure during experiments. By studying structural responses numerically, it is also possible to extract results that can be important in order to identify different properties of a real structure or specimen in an experiment.

The studies that were carried out in this thesis are in general based on constitutive models where concrete is represented by a combination of damage and plasticity. The choice to represent concrete during highly dynamic events through the use of a damage-plasticity model is a continuation of previous research at Chalmers the University of Technology, Nyström (2013), and the University of Glasgow, Grassl et al. (2013), and has been shown to successfully describe loading and unloading of concrete with large confinement effects and for the propagation of shock waves.

1.4 Scope and limitations

The thesis focuses on normal concrete and reinforced concrete. One of the studies investigates the development of spalling damage in a concrete wall and the influence of different material responses of the concrete during non-monotonic loading and unloading. The second study investigates the performance of the material model CDPM2, Grassl et al. (2013); Nyström (2013), which is a combined damage and plasticity model. Both studies are numerical studies. For the first study, investigating spalling, experimental data that can be used to verify the hypothesis have not been found. The second study, investigating the performance of CDPM2, is a numerical study of experiments designed and performed by other researchers. The tests were not designed to be primarily used to evaluate the response of a specific material model, but rather numerical models in general.

2 Loads due to blast and impact

2.1 Overview

Loads are usually divided into static loads, quasi static loads and dynamic loads based on the time duration of an action. However, dynamic loads span over a range of time intervals. Different time frames yield different types of responses, Gebbeken, Greulich, and a. Pietzsch (2001), both for the material response and the response of the whole structure and, thus, create different demands for the analytical routines and material representation.

Dynamic response can occur within different time frames. For example, when oscillation occurs in a structure, the time duration can be seconds or parts of seconds. During this time frame, the deformation of the structure changes and, thus, the internal and external forces for the structure change. If a moving object hits a structure, the response will depend on both the velocity and the material properties of the two bodies, Leppänen (2012). When a high velocity impact between a small object and e.g. a beam occurs, such as a fragment impact, local effects in the form of damage around the zone of impact will develop within a much shorter time interval than the global bending deformation of the beam.

When the type of loading condition is classified based on time intervals, a common measure used is the strain-rate caused at different martial points in the structure. By measuring the change in strains per time unit it is possible to separate long time intervals from short time intervals. In Figure 2.1 different engineering applications with regards to strain-rates are shown. The picture is taken from Nyström (2013) and is based on a variety of publications: Bischoff and Perry (1991); Field et al. (2004); Gebbeken and Ruppert (2000); Ramesh (2008); Zukas (2004). In this thesis responses of concrete structures due to blast waves from the detonation of explosives or the simulation of such event are studied. This corresponds to the "blast and impact" region in Figure 2.1.

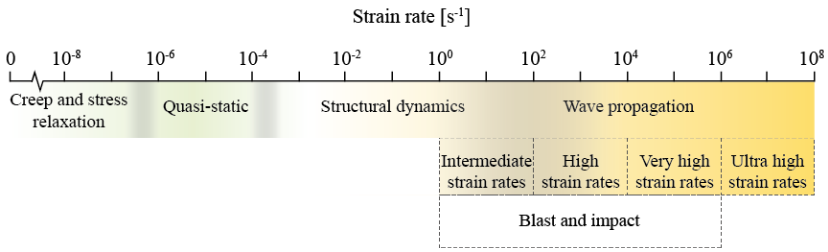


Figure 2.1: *Strain-rates and associated problem aspects taken from Nyström (2013)*

2.2 Blast wave in air

When a highly explosive substance detonates in the open air, a sudden release of energy occurs. An explosion can be characterised as a sudden volumetric expansion of matter due to physical or chemical change of state. The change of state results in a sudden release of potential energy to mechanical work. For a detonation of an explosive substance the expansion of gases creates an overpressure which generates mechanical work when the surrounding air is forced away. The highly compressed air surrounding the detonation creates a blast wave that propagates with super sonic speed from the epicentre of the explosion. Directly behind the front of the blast wave is a region where pressure, temperature, density and particle velocity are distinctly higher than the surrounding air. When the blast wave moves away, these properties rather rapidly return to their original states. For a fully developed blast wave, the pressure rises from the normal atmospheric pressure to a peak pressure more or less instantaneously. The time for the pressure to rise is therefore generally considered to be a singular jump in time. The pressure then decreases exponentially until an under-pressure is reached that returns to the original atmospheric pressure; see Figure 2.2. In structural analyses and evaluations, the blast wave is usually simplified and only the overpressure is considered. Thus, no negative pressure is applied to the structure. Furthermore, the exponential pressure decreases after the shock wave has reached the structure and can also be simplified to a linear decrease, Johansson (2012).

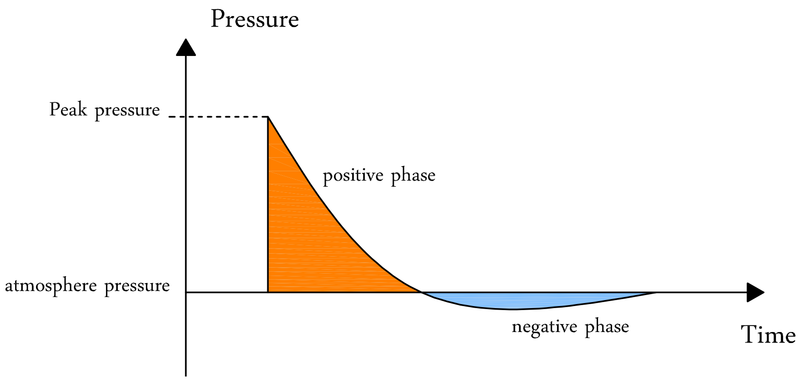


Figure 2.2: *Principal pressure-time relation for a blast wave in air*

2.3 Fragments

The detonation of a bomb will create not only a blast wave but will also send away fragments, Leppänen (2004). The detonation of the explosives will rip the casing of the bomb into small pieces creating a cluster of fragments, Janzon (1978). The fragments will represent mass and velocity and can add a substantial load to a structure. A blast wave and a cluster of fragments can reach a structure at different time intervals because of differences in which speed they travel, Nyström and Gylltoft (2009). Even though the actual impulse of a cluster of fragments might be distinctly smaller than from the blast wave, the damage to the surface of a structure can be substantial, Nyström and Gylltoft (2009). Furthermore, the damage to the surface may lead to a loss of capacity and a stiffness influencing the overall structural response. A study of the combination of fragments and blast wave showed a synergy effect, Nyström (2008). It was established that the combination of fragments and blast wave created a larger deformation than if the deformations due to the fragments alone were added to the deformation due to the blast wave.

An important difference between the stress response in a structure due to a blast load and a cluster of fragments is that the stress towards the surface of the structure is more or less independent of the properties of the structure for a blast wave. For the cluster of fragments, the material properties of the fragments and the material properties of the structure will affect the stress wave that will propagate through the structure after the impact. The velocity of the fragments will be important for the magnitude of the stress wave but the stiffness of the fragments as well as the stiffness of the material in the structure will also influence both the magnitude of the stress wave and the duration of the stress wave, Leppänen (2004); Leppänen (2012). The effect of the material properties of fragments can be indirectly studied by looking at the response of flyer plate impact tests, Grady (1996); Gebbeken, Greulich, and A. Pietzsch (2006); Riedel et al. (2008), where the material properties of the materials involved are used to calculate the material states.

3 Fracture due to blast and impact

3.1 Overview

A structure that experiences loading due to an explosion undergoes, in general, different reactions at various stages of the loading and unloading. The most obvious differences are responses related to local and global response, see Figure 3.1. However, for dynamic loading another important factor is the stress of the structure during different time intervals. Global failures can, in general, be determined based on the total energy or work applied to the structure from the load compared to the ability of the structure to absorb externally applied energy, Johansson and Laine (2012).

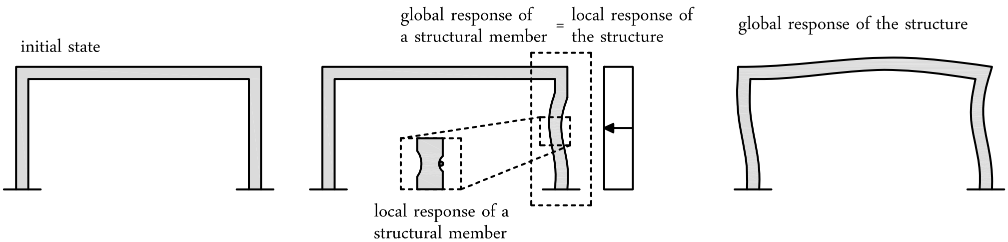


Figure 3.1: *Local and global response of a structure structural member and of an entire structure*

Other fractures are determined by stresses caused by the change in applied load over time, $\frac{\delta load}{\delta t}$, Meyers (1994). Thus, the total intensity of the load is not necessarily critical for initiating a failure process. If a certain load condition is critical for a specific fracture, the corresponding time interval when that fracture occurs has to be studied. The time frame for a local failure, such as spalling or scabbing, is shorter than for a global failure, such as a flexural failure or shear failure.

3.2 Description of failures and what can cause them

In the case of dynamic load conditions, different failures can occur in a structure compared to static load conditions. In the following section(s), the general conditions and related time intervals for different dynamic failures are presented. Dynamic failures are named differently in various literature. Here, the names of the fractures correspond to U.S. Army Corps of Engineers et al. (2008).

3.2.1 Spalling

When a wall or a slab is subjected to a highly intense blast wave or a high-speed object, such as a fragment or projectile, a compression wave may occur inside the structure. When a compression wave reaches the free edge at the back side of the structure a stress reflection occurs. This reflection can also be seen as a release wave due to the state of equilibrium; see Paper A. When a release wave starts to propagate back towards the loaded side of the structure, the stress state at the boundary will be zero. However, if the compression stress further into the structure has decreased due to the characteristic of the applied load, the release wave can create stress states made up of tensile stresses. If these tensile stresses are large enough they can result in a crack initiation in the structure, Mcvay (1988); Meyers (1994). If the applied load is intense enough, fragments of the structure can spall off at the back side of the structure which is not directly subjected to the load, see Figure. In a brittle material, such as concrete, spalling can occur more easily due to the large difference between compression and tensile capacity. In Paper A, spalling in concrete subjected to a shock wave blast was studied for a simplified one-dimensional specimen. It was shown that for certain cases of spalling, the cracks can develop during multiple cycles of stress waves rather than instantly as soon as the tensile strength is reached, which is generally argued by Mcvay (1988); Meyers (1994); Leppänen (2012). If this type of response is aimed at describing in numerical models, the constitutive laws describing the concrete become important with regards to non-monotonic crack propagation.

3.2.2 Scabbing

Spalling is generally associated with close-in detonation or clusters of fragments. A close-in detonation can create pressures strong enough to cause crushing and cratering on the loaded side of the concrete. Since the damage is caused by close-in detonation, the pressure confinement effects increase when the pressure propagates further into the structure but the pressure will also disperse when the wave propagates. Thus, the extension and depth of the cratering is usually limited.

3.2.3 Flexural and shear failures

The flexural response of a reinforced concrete structure subjected to dynamic loading is similar to the flexural response of a statically loaded structure. Failure modes are either crushing of the concrete or yielding of the reinforcement. Furthermore, different anchorage failures and support slipping can occur. Shear failure can occur in a similar manner as for statically loaded structures. However, for very intense loads shear failure can occur in earlier stages of the evolution of the deflections. When the load is first applied, a beam or a slab has a rigid body motion before the support forces develop. The internal forces in the structure occur first when the support forces appear. Thus, bending moments and shear forces emerge from the supports and propagate from there, Andersson and Karlsson (2012). Highly intense loads can therefore result in stresses on the structure closer to the supports compared to the same structure subjected to static loads. Direct shear, Krauthammer (2008), is a failure that may appear if the stresses in the structure are great early during the structural response.

4 Numerical analyses of failure in concrete structures

When a material like concrete is to be modelled through numerical analyses a number of approaches to describe material fracture are possible, such as, discrete crack models, non-local crack models and smeared crack models, Jirásek (2010). Any chosen approach has advantages and disadvantages. The work carried out in this thesis is based on a smeared crack approach where the properties of the inelastic deformations of concrete is smeared out over a band width. The material model studied in detail in this thesis was developed during a previous PhD project by Nyström (2013).

4.1 Essential properties of concrete constitutive models

Depending on the aim of a structural model and the loading condition, the demands on the utilized material models vary. When the response of, for instance, concrete is studied during static conditions, there are various aspects that need to be considered. Different stress states will yield different demands on the complexity of the material model. If a material model is simplified, it also, becomes more limited in terms of general applicability.

In dynamic response there are aspects of material response that are unique or of greater importance compared to static conditions. One of the unique properties, only implicitly considered in dynamics, is how the material strength depends on the rate of applied stress, generally measured as the strain-rate, $\frac{\delta\varepsilon}{\delta t}$. Other features which are less common in static evaluation include non-linear compaction under hydrostatic compression and residual strength after extensive material damage. These final two properties are more likely to occur in fast dynamic events because of inertia effects creating local confinements during very short time intervals. There are many different approaches to describe these responses with the help of a constitutive model. This thesis is a continuation of a previous project where a combined damage and plasticity model was developed in order to describe these properties, Nyström (2013); Grassl et al. (2013). The evaluation of properties is presented in Nyström (2013) and in Figure 4.1 where the most important features of a material model describing concrete are placed at the base of the pyramid and in decreasing importance towards the top.

In Nyström (2013) strain rate dependency of strength, post-peak softening, non-linear compaction curve for high pressures and residual strength in confined compression were concluded to be the most important features of the response of concrete for dynamic loads. Post-peak softening is a material property which is not unique to dynamic conditions. For multi-axial loading, pressure-sensitive strength is also considered to be a very general property of concrete.

Strain-rate dependent strength, however, is an effect which only impacts the mate-

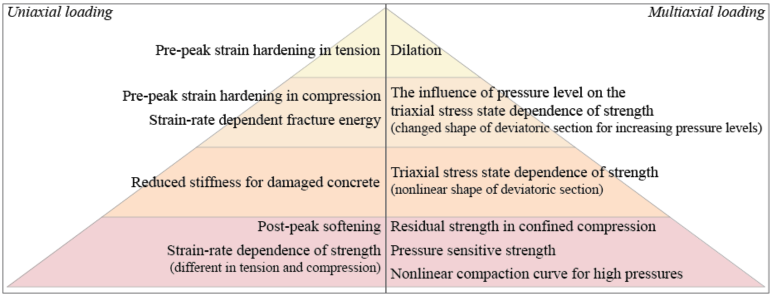


Figure 4.1: *Assessment of properties during development of material model, from Nyström (2013)*

rial properties during dynamic conditions. The dynamic tensile strength has been shown to be as high as eight times the tensile strength for a static load condition, Malvar and Ross (1998). In compression, the observed dynamic strength is about 2,5 times the static strength, Bischoff and Perry (1991).

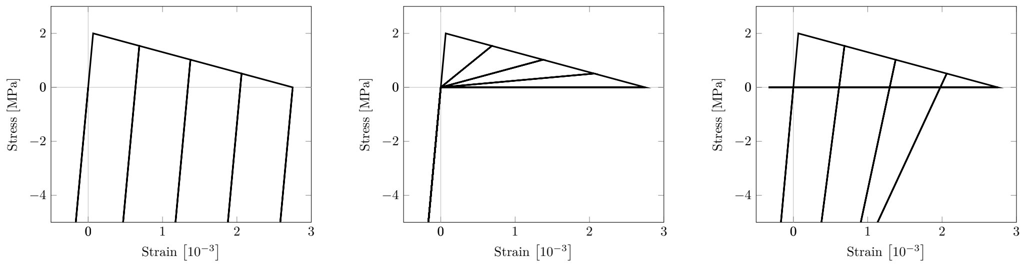
Residual strength in confined compression and the non-linear compaction curve for high confinement pressure, Williams et al. (2006), are aspects of the response which occur both during dynamic and static conditions. However, high confinement pressures are difficult to create and maintain during static conditions since the confinement requires externally applied loads. For dynamics, the confinement can be created due to inertia effects in the material where a high pressure is confined by the surrounding material.

4.2 Constitutive models for concrete fracture in tension

Concrete, with its very different properties depending on the stress state, needs rather complex constitutive laws to capture the response for an arbitrary stress state. In many applications, the focus lies on the ability to capture a specific response, which allows for reducing the complexity of the material model.

If a concrete specimen is loaded in monotonic tension under static condition, the tensile stress will reach a peak value from which the stress gradually decreases with increasing deformation. This material softening develops more or less exponentially but in most applications, it is considered sufficient to use a bi-linear softening law, Gylltoft (1983), or even a linear softening law.

In Paper A, spalling in a concrete wall subjected to a close-by explosion is studied. In this paper, the tensile response of the concrete is modelled according to three different constitutive laws, a plasticity model, a damage model and a combined damage and plasticity model. All of them use a linear softening when cracks are developed in the concrete. When a crack is forming, even though the stress-strain response is similar, the structural response becomes different. A plasticity model and a damage model can describe the same type of softening in the material. However, in a plasticity model, the elastic response upon unloading will be based on the original stiffness of the material whereas, for a damage model, the stiffness will be reduced. This means that the final structural response can differ if, e.g., a non-monotonic response takes place, as shown in Paper A. Since the concrete undergoes multiple stages of compression and tension, see Figure 4.2, the response during the transition will affect the result.



(a) *Plasticity* (b) *Damage* (c) *Damage-Plasticity*

Figure 4.2: *Stress-strain relation for cyclic tensile and compressive loading of concrete for (a) aplasticity model (b) a damage model (c) a damage-plasticity model*

4.3 Evaluation of CDPM2

One of the aims of this study has been to evaluate and confirm the potential areas of development for the new material model CDPM2 when used in dynamic models and to identify possible areas of improvements. In Nyström (2013), the following features were suggested as potential areas of improvement:

- Increased material stiffness after non-linear compaction
- Avoiding of overestimated fracture energy due to double strain-rate dependency
- Reduction of the strain-rate dependency for compressive strength for modelling with solid elements at high strain-rates

The aim was to study these features in Paper B. The conclusions from the study is summarised below:

Increased material stiffness after non-linear compaction

The constitutive laws for CDPM2 do not affect the material stiffness after volumetric compaction, i.e., when the concrete is compressed in a way so that the pore system collapses and a more solid material is created. This feature of the concrete material is relevant when a structure is hit by a high speed projectile or fragment. It was shown in Nyström (2013) that CDPM2 had too low a stiffness for hydrostatic pressures of 2 GPa and higher. In Paper B structures subjected to blast loads resulted in flexural deflections was studied and for that type of response, such high pressures were not reached.

Avoiding of overestimated fracture energy due to double strain rate dependency

The constitutive laws that treat the tensile strength and the strain softening branch in tension results in an increased tensile strength and an equally increase of the maximum crack opening when zero stress is reached, see Nyström (2013) and Paper B. Since both the strength and the strain that represent the maximum crack opening are increased, the dynamic increase factor for the fracture energy, DIF_{G_F}, is equal to the dynamic increase factor for the strength squared, $DIF^2_{f_{ct}}$. The available studies of how the strain-rate influence the fracture energy are limited but indicate that the dynamic increase factor for fracture energy is proportional to the rate effect on the tensile strength, Weerheijm and Van Doormaal (2007). In Weerheijm and Van Doormaal (2007), it is also concluded that the maximum crack opening remain constant. The study in Paper B shows that a variation of input values for the fracture energy results in a large variation of deflection for beams and plates. Therefore, an overestimation of the strain-rate effect for the fracture energy can be expected to result in an underestimation of deformation similar to those seen in Paper B.

Reduction of the strain rate dependency for compressive strength for modelling with solid elements at high strain rates

The strain-rate dependency of the material strength is, in CDPM2, treated by using a magnification factor for the concrete strength based on the strain rate of the total strain, Nyström (2013). For tension, the rate factor is based on the expression proposed by Malvar and Ross (1998) and for compression, the expression proposed in *fib Model Code for Concrete Structures 2010* (2013) is used. However, in Nyström (2013), it was shown that the expression proposed in *fib Model Code for Concrete Structures 2010* (2013), combined with 3D continuum elements, resulted in an overestimation of the rate effects for the compression strength. Paper B studied the flexural response of a beam and a plate with and without the strain rate effect turned on for CDPM2. It was shown that the strain rate effect has a significant influence on the deflection of the structure. However, how much of that effect that comes from the overestimation of compression strength and how much that comes from the overestimation of the fracture energy could not be determined in the study. However, to treat the element size dependency affecting the compression strength, shown in Nyström (2013), the expression used to describe the strain rate dependent compression strength should be updated so that the inertia effects are omitted from the constitutive laws for 3D continuum elements.

Strain softening for compression failure

Concrete failure in uniaxial compression is reached after a softening, Karsan and Jirsan (1969). In a smeared crack approach, the localisation zone where the inelastic deformation takes place needs to be defined in order to obtain a correct stress-strain relation, Jirásek (2010). The implementation of CDPM2 defines a stress-strain relation based on a pre-specified element size, and the ductility measure, Grassl et al. (2013). In Paper B, problems with convergence occurred for the slab analyses. It was believed to be due to the description of the softening in compression. To treat this, elastic elements were used at the concrete surface by the supports and at the top of the slab where the plastic hinge occurred.

5 Conclusions

Two studies have been presented in this theses. The first in Paper A, where spalling in a concrete specimen due to a blast wave was studied. The assumption that the damage due to the tensile stress from the release wave develops instantaneous within a time singularity was shown to be inaccurate. During the development of the fracture, e.i. the crack propagation, the release wave continued to propagate past the point of first crack initiation. In the studied case where the length of the blast wave was much longer than the studied concrete specimen, the crack did not form until multiple compression and release waves had passed. It was also shown that the choice of constitutive model to describe the tensile fracture affected the response and the propagation of blast wave. It is therefore concluded that for some case of spalling, the choice of constitutive model for concrete tensile fracture can be crucial.

In the second study, presented in Paper B, the performance of the new material model, CDPM2, based on combined damage and plasticity, was investigated for blast loaded beam and slab specimens. Some of the limitations presented in Nyström (2013) and how they affected the response of beams and slabs were investigated. It was concluded that the strain rate dependency of strength and fracture energy of the concrete affected the response of the structures. It was also shown that the chosen fracture energy influenced the deflection of the structures more than if strain-rate dependent strength was included for the concrete. Thus, it was concluded that the strain-rate dependency of the fracture energy must be described correctly. The available research with regard to strain rate dependency for the fracture energy, Weerheijm and Van Doormaal (2007); Schuler et al. (2006), suggested a lower increase of fracture energy due to increased strain-rates compared to tensile strength. The constitutive laws in CDPM2 should, thus, be adjusted to better fit available data.

Finally, in Paper B, convergence problems occurred in areas of large compression and to some extent distortion for the slab analyses, e.i. around the support and at the top of the slab. To treat this in the study, elastic elements were used in the outer layer of the slab in these areas, see Paper B. It was assumed that the convergence problems were due to the description of the strain softening for CDPM2 in compression and that the concrete showed a too brittle behaviour. Therefore, the localisation length should be possible to define for an analysis in the same way as for tensile fracture.

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Appended Papers A–B

Paper A

Spalling in Concrete Subjected to Shock Wave Blast

J. Ekstrom, R. Rempling, M. Plos.
Submitted to "Engineering Structures".

Spalling in Concrete Subjected to Shock Wave Blast

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Abstract

This paper studies the response in a concrete wall subjected to shock wave blast, leading to spalling failure. This situation is important since spalled-off fragments in protective structures may cause severe injury to the persons or equipment it is supposed to protect. Previous research indicates that spalling occurs when and where the tensile strength of a strain-softening material like concrete is reached. By using a simple uni-axial numerical model, this study shows that spalling instead occurs when the cyclic response from a blast wave gradually increase the inelastic strains in the concrete. This means that spalling takes place after several loading cycles and not necessarily at the depth where tensile strength is firstly reached. Furthermore, the study shows that the cyclic response in the material model used for numerical simulation has a decisive influence on the position and extent of the resulting spalling crack.

Keywords: Spalling, stain-softening, wave propagation, concrete, cyclic response, transverse/reverse loading, dynamic response, cyclic crack propagation

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1. Introduction

1.1. Background

In society, there is a certain need of preparedness for different emergency situations, such as unprovoked explosions within the urban community or acts of war. In these situations, important buildings, such as civil defence shelters and protective structures are fundamental to our readiness. In Sweden, the building stock is ageing and the focus is upon maintaining existing buildings rather than demolishing and rebuilding them. Therefore, the ability to evaluate and strengthen existing buildings exposed to new demands is of major importance.

A structure can experience different types of load effects depending on the size and position of an explosion. During shock dynamics - such as blast and impact loading - the time frame during which the structural response takes place is relatively brief. In structural evaluations related to static conditions, the time frame from unloaded structure to fully loaded structure can span over a few seconds, i.e. short term load, and up to hours or

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years, i.e. intermediate and long term load. For shock dynamics, the time duration from the load origin to the peak stress of the structure can be as brief as a fraction of a millisecond [1, 2]. During this time span, the structural response differs greatly from a static load case and evaluation procedures used for static conditions are in many cases invalid [3]. Fracture modes which never occur for a static load can turn out to be the most critical response [4, 5]. However, in both static and dynamic response, similar structural and material properties are considered favourable, such as increased ductility, both in terms of structural response [6] and material response. The rationale for this is that a high ability to withstand large deformations can give the structure a fine ability to redistribute forces and for dynamic loads to absorb the high energy content of the load as well.

Concrete is a material which undergoes softening during fracture, both in compression [7] and in tension [8]. A common way by which the ductility may be increased, compared to plain concrete, is to add fibres so that the energy dissipation during fracture of the material increases [9]. For this reason, it may be argued that fibre reinforced concrete, when used in structures, provides an increased ability to withstand high dynamic loads compared to a structure designed with plain concrete. Many experiments and numerical analyses [10, 11, 12, 13] indicates that structures of fibre reinforced concrete have a higher resistance to damage critical to the internal function of individuals, such as pieces of concrete thrown into the interior of the protective structure. The latter phenomena of concrete failure, with cracks parallel to the surface, is called spalling [14, 15, 16].

Numerical models can be powerful tools to understand the behaviour of a structure subjected to high dynamic loads. Different parameters of the load, the geometry and the material can be evaluated and the structural behaviour can be studied from the time the load is applied until the final state of the structural response. For comparison, for experimental studies the final stage of the response is usually the only state which can be studied in detail. Hence, by modelling experiments, a better understanding of the entire response sequence can be achieved.

1.2. Problem identification

In order to protect the inhabitants of a building from an explosion, it is important for the material not to spall when subjected to a blast or fragment load. Spalling can occur when a compression wave with a negative pressure-time gradient reaches a free surface from where a release wave will start to propagate in the opposite direction [14, 15, 16]. The pressure-time gradient of the applied load will cause a tensile stress when the release wave propagates back into the structure. If the tensile stress were to reach the tensile capacity, damage would be initiated. Earlier studies have assumed that spalling occurs when the tensile capacity is reached, e.g. [14, 15]. The stress state in a wall where a plain pressure wave with a linearly decreasing pressure has propagated to the unloaded side is illustrated in Fig. 1. In the same figure, the stress state is shown when the release wave that propagates back towards the loaded side has reached a point where a tensile stress equal to the tensile strength has been reached .

The approach assumes that a fully open crack in the material is created during the stress-time singularity which defines the pressure and release waves when the tensile capacity is reached. The assumption that a fully opened crack can be created during this singularity was supported in a study of wave propagation in strain-softening materials [17]. However, for a strain-softening material the momentary drop in tensile stress in the crack means that the deformation discontinuity, i.e. the crack, is created during a time singularity. Thus, the energy within the stress wave must be infinite. Since the pressure peak and, thereby, energy of the pressure wave is finite, the crack cannot open during the time singularity. In fact, a crack can only be initiated during the singularity but the crack can only develop during the time after the wave has passed the material point. To achieve the response described in [14, 15], the crack initiation and the development of the crack needs to be neglected. Therefore, this approach cannot consider the effects of the material ductility. Instead, using this approach, varying ductility of the material will predict the same response. Thus, a design using a more ductile material, such as fibre reinforced concrete, will not be favourable compared to a brittle material, such as ordinary plain concrete, if the material were to undergo strain softening. However, this is in general not the conclusion from experiments where structures with fibre concrete experience less damage compared to the same structure with plain concrete [10, 12].

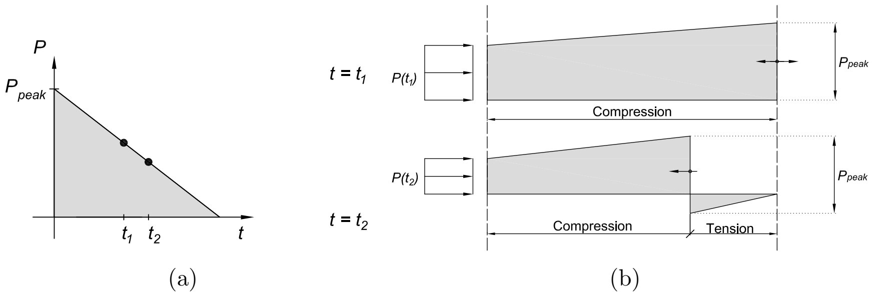


Figure 1: Illustration of (a) the applied pressure wave towards the left edge of the wall and (b) the stress distribution through the thickness of the wall when the pressure wave reaches the unloaded right edge of the wall (above) and at crack initiation (below).

1.3. Aim and objectives

The aim of the study was to present how material mechanisms influence the crack propagation in brittle material subjected to an impulse load which result in spalling damages. The objectives were to:

- identify the general response of crack-softening materials, such as concrete, with regard to crack propagation during spalling attributable to impulse loading,
- investigate the effect of material strength and crack-softening during the fracture process in brittle crack-softening materials according to the brittle crack spalling approach described in [14, 15],
- present the effects of using a plasticity model, a damage model, and a combined damage and plasticity model to describe the material response of spalling concrete,
- identify important properties of the constitutive models used to represent a material which aims to describe spalling phenomena in brittle materials such as concrete.

1.4. Method

The analytical solution to crack initiation for spalling damage in a crack-softening material was studied for a one-dimensional wave propagation. The effect of including the crack-softening, i.e., the fracture energy was investigated, thus, the fracture was not allowed to emerge within a defined singularity. Furthermore, the crack-softening influences on crack propagation and the development of stress waves in the structure were studied with the help of this approach.

A one-dimensional numerical model for uniaxial response was developed to study the effect of strain localisation and the nonlinear non-monotonic material response for a section of a concrete wall subjected to blast waves. Different types of models were used to represent the softening of the material and to study the importance of choosing a correct representation of the non-monotonic material response during crack-softening. The crack-softening was represented by a strain-softening approach [18]. The crack propagation and the position of the fully open crack were compared to the predicted response using the simplified approach for brittle crack spalling according to [14].

1.5. Limitations

In this paper, the conceptual differences between a brittle crack approach and a strain-softening crack approach for the modelling of spalling are presented. The material parameters and load values have been chosen in order to highlight the consequences of the different approaches. The shape of the time-pressure response of the load and the strain-softening were simplified using linear relations. Further, strain rate effects that might influence the material tensile and compression strength [19, 20], fracture energy [21, 22] and Young's modulus [23] were not considered. Consequently, the case studied does not represent a specific experiment nor load case. Most experiments on the subject are usually close range detonations and are thus more complex. These types of experiments require the use of a three-dimensional model to be correctly represented in a finite element analysis. However, the blast load in this study generates a pressure wave which appears as plain in the wall section investigated.

2. Brittle crack spalling

A simple approach to determine whether spalling may occur in a concrete wall is the evaluation method presented in [14, 15]. In this paper, the approach described in [14, 15] is referred to as the "Brittle crack Spalling Approach". The principle of that hypothesis is that spalling cracks appear where a tensile stress, according to a linear elastic model, exceeds the tensile capacity. Due to the defined singularity of the pressure wave, a stress state of a material point may momentarily change from a state of compression to a state of tension. Therefore, if the tensile capacity were to be exceeded, a fully open crack is assumed to appear during the singularity, thereby creating a new free surface at each side of the crack. From this new free surface, the same phenomena which caused the first spalling crack to appear can create a second spalling crack. This happens when a new tensile release wave propagates further into the wall from the new free surface which is subjected to a state of compression. The stress state at the point of the first spalling crack is illustrated in Fig. 1.

2.1. Limitations of applicability of concept

The most obvious limitation of the brittle crack spalling approach for material with strain-softening, as described in [17], is that the shape of the softening branch and the ductility of the softening become irrelevant since the strain reaches infinity within the stress-time singularity. However, given the approach chosen in [17], infinite strain is only achieved if the extension of the crack are to decrease to an infinitesimal value. Given this approach it is no longer possible to relate a stress to a unique inelastic strain. Furthermore, due to its singularity, the elastic phase the material has to undergo before fracture will yield infinitesimal strain energy and may, according to [17], be neglected. However, even if the strain softening can be neglected the model needs to hold for any strain-softening branch and, consequently, for a perfectly brittle material without any fracture energy. However, if the fracture energy were zero, even infinitesimal strain energy achieved before fracture cannot be neglected since the material possesses no ductility. Furthermore, as described in Section 1.2, the results predicted by this approach can only be achieved if the material is considered without fracture energy and the fracture process is hidden in the defined stress-time singularity.

2.2. Important aspects of the study

An important aspect of this study was to investigate the crack propagation during the spalling of concrete and how the fracture energy influence the results and wave propagation. To describe spalling given certain load conditions, the material response to cyclic loading during crack propagation becomes important. The study investigated whether a simple numerical model can describe spalling and how different ways to model material response to cyclic evolution of the crack propagation influence the response, such as plasticity and damage models (Fig. 2).

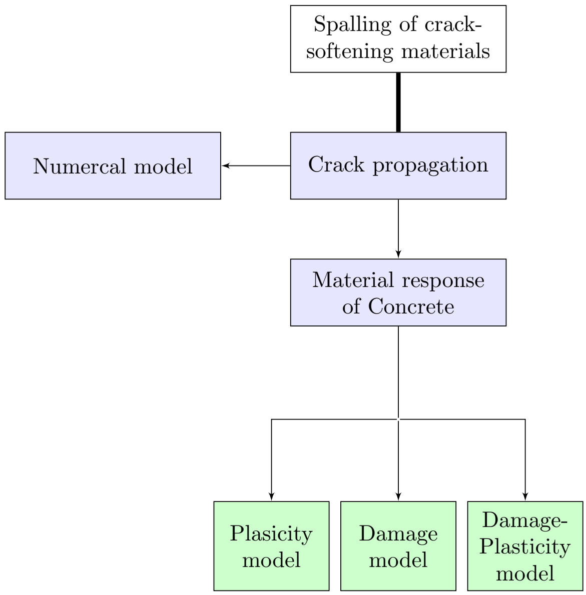


Figure 2: Important aspects of the study and their mutual dependency.

3. Numerical study of spalling in strain-softening materials

3.1. Overview of study

Wave propagation and spalling in a plain concrete wall element were studied with the help of a numerical one-dimensional (1D) model in the thickness direction of the wall shaped as a concrete rod. The thickness of the wall studied was chosen to $L = 300mm$. The applied load was simplified to a plain pressure wave with linear decrease of the pressure after a momentarily reached peak pressure (Fig. 1a). The load was chosen in order for spalling to be achieved, both in the simplified theoretical approach, [14], and in the numerical model used in the study. The load corresponded to a hemispheric charge of 1500 kg TNT at a distance of 5 m calculated according to [24]. Geometry and load data are presented in Table 1. The Young's modulus was not increased due to geometric confinement; thus, a plain stress state was assumed.

The structural model used was an explicit nonlinear dynamic model with bar elements of proportional stiffness damping based on a linear elastic stiffness matrix. The analyses were carried out using MATLAB®.

3.2. Description of material characteristics

The concrete was approximated as an elastic material until the tensile capacity was reached; thereafter, a linear softening decreased the tensile capacity for increased strains.

Table 1: **Geometry and load**

Parameters	Value
Wall thickness [mm]	30
Equivalent TNT charge weight (hemispheric expansion) [kg]	1500
Distance to wall structure [m]	5
Maximum aggregate size [mm]	16
Peak pressure [MPa]	50.74
Load duration [ms]	1.31

Table 2: Material parameters

Parameters - general	Value
Young's modulus [GPa]	30
Tensile strength [MPa]	2.0
Fracture energy [N/m]	132
Maximum aggregate size [mm]	16
Density [kg/dm^3]	2.35
Plasticity model	
Softening law	Linear
Damage model	
Softening law	Linear
Damage-Plasticity model	
Softening law	Linear
Hardening modulus	$0.01E$

The material was modelled according to a smeared crack approach to describe the crack localisation [18]. The spalling of plain concrete was studied for three different material models for concrete, a plasticity model, a damage model and a combined damage and plasticity model [25]. The softening law was defined to resemble plain concrete with linear softening. The fracture energy was calculated according to [26]. The main difference in the material response between the three models was the response during unloading after crack initiation. The material parameters used are presented in Table 2. In compression, the response of the concrete was linear elastic; plastic strains or damage propagation were not allowed to take place. In tension, linear strain softening was assumed upon crack initiation.

In the plasticity model, the strain softening was achieved with the help of using a negative hardening variable. For stress states below the reduced tensile capacity, the response remained elastic with a stiffness based on the Youngs modulus (Fig. 3a). In the damage model the linear softening was achieved through an isotropic damage evolution during which the stiffness and stress had been scaled down from a linear elastic response. Damage affected the stiffness during unloading for positive strains but the stiffness was kept undamaged for negative strains (Fig. 3b). The combined damage and plasticity model consisted of an elastic-plastic model with hardening and was combined with an isotropic damage model to achieve linear softening. The hardening modulus was chosen according to [27]. The damage evolution was based on the plastic strain and affected the stiffness both during the unloading for positive strains and the loading for negative strains (Fig. 3c).

3.3. Numerical approach

The development of the spalling damage in the wall was studied with the help of a nonlinear numerical model based on the Central Differential Method. For each new time step, the displacement vector was determined [Eq. (1)]. The displacement vector was used to determine the strain increments in the elements [Eq. (2)]. The stresses in the elements and thereby internal force vector was calculated by the material routine [Eq. (3)]. With the displacements and internal force vector known, the acceleration vector and velocity vector could be solved by combining Eq. (4) and Eq. (5). Displacement, velocity and acceleration together with the material response were used as inputs for the next time step along with the external load vector. The results evaluated in this study are the strains and stresses from the material model and the node velocities. The scheme of the numerical routine is presented in Fig. 4.

The following equation was used to go through the numerical routine described in Fig. 4:

$$u_{n+1} = u_n + h \cdot \dot{u}_n + \frac{h^2}{2} \cdot \ddot{u}_n \tag{1}$$

$$\Delta\varepsilon_{n+1} = \frac{u_{n+1} - u_n}{L} \tag{2}$$

$$q_{n+1} = q_n + \Delta q_{n+1}\left(\varepsilon_n, \Delta\varepsilon_{n+1}\right) \tag{3}$$

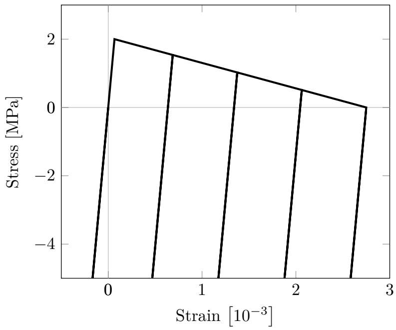
$$M\ddot{u}_{n+1} + C\dot{u}_{n+1} + q_{n+1} = F_{n+1} \tag{4}$$

$$\dot{u}_{n+1} = \dot{u}_n + \frac{h}{2} \cdot \left(\ddot{u}_n + \ddot{u}_{n+1}\right) \tag{5}$$

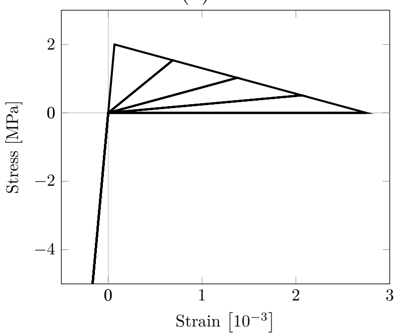
where u is the displacement, $\dot{u}$ the velocity, $\ddot{u}$ the acceleration, q the internal force and h the time interval.

3.4. Finite element analysis

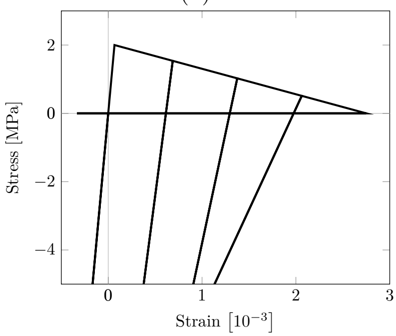
Bar elements were used to model the concrete wall and were preferable to a mass spring system in which lower eigenvalues would be achieved resulting in the need to use a higher damping of the system. Geometric nonlinearity was not considered for the elements. The total number of elements was chosen at 100 elements. To describe the softening response of the concrete during crack propagation, a strain-softening approach was chosen. The representation of a crack was smeared out over a fracture zone equal to three times the maximum aggregate size [22]. With an assumed maximum gravel size of 16 mm, the crack



(a)



(b)



(c)

Figure 3: Stress-strain relation for the different material models, (a) plasticity, (b) damage and (c) damage-plasticity, during non-monotonic material response.

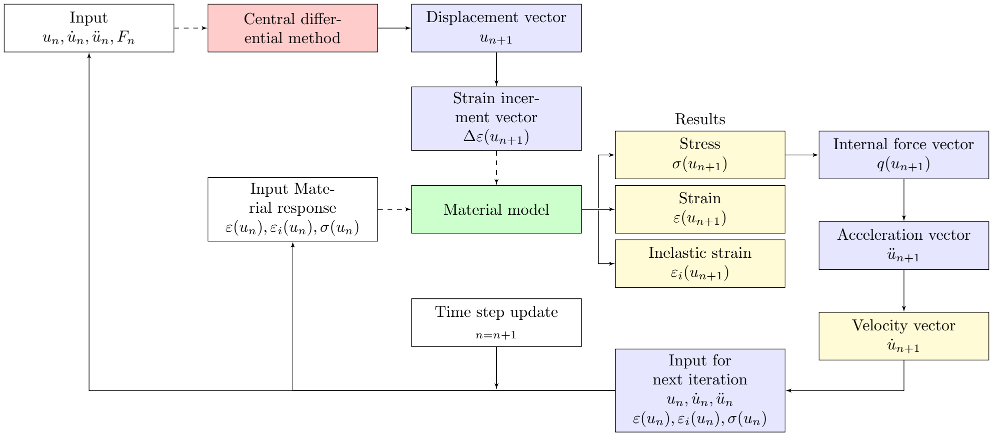


Figure 4: Scheme of numerical routine

bandwidth became 48 mm. Thus, to achieve a proper representation in the numerical analyses, the inelastic strain needed to be distributed over this distance. If a larger zone of inelastic strains were achieved more than one crack would be represented in the result, i.e. more than one crack would appear. If the inelastic strains were distributed over a smaller zone than 48 mm, the model would represent a more brittle material than what was anticipated.

The mass matrix of an element is described by Eq. (6). The damping of the system was based on proportional stiffness damping according to Eq. (7) and Eq. (8). The linear elastic stiffness matrix was used for all states of the system and all time steps throughout the analyses to define the damping matrix. Thus, the mass matrix and damping matrix of the system were assembled once before the first time step and were then used throughout all time steps. The numerical model and input for the model geometry, material model and external load applied on the structure are presented in Fig. 5.

$$M_i = \frac{\rho_i A_i L_i}{6} \begin{pmatrix} 2 & 1 \\ 1 & 2 \end{pmatrix} \tag{6}$$

$$C_i = a K_{0,i} \tag{7}$$

$$a = \frac{2\xi}{\omega_{max}} \tag{8}$$

where ω_{max} is the highest response frequency of the whole system and ξ was chosen to 1.

3.5. Parametric study of spalling of softening materials

In order to show the importance of the non-monotonic response of a strain-softening material during crack propagation and spalling, three different model approaches were com-

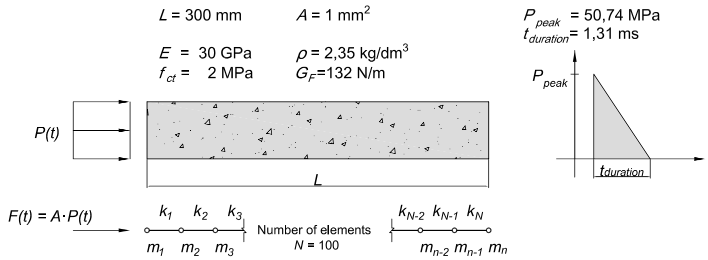


Figure 5: Geometry, boundary conditions and load.

pared. These three different approaches yield the same strain softening in tension. However, the response where crack propagation appears during non-monotonic material response vary among the three approaches. The same load case and structure geometry were used for the three models and the crack propagation during the duration of the load was studied along with the final crack distribution of the analyses.

4. Results for study of spalling in softening materials

4.1. Overview of results

The results from the analyses are presented with respect to crack propagation for different time steps throughout the analyses. The crack propagation along the normal section through the wall is presented and compared with the predicted position of the first spalling crack according to the brittle crack spalling approach, [14]. For the positions where the largest crack openings were achieved in the different analyses, the stress-strain response is presented to show the development of the crack growth and the non-monotonic response of this material point. Finally, the distribution of velocity along the bar is studied in order to determine the final spalling velocity achieved.

4.2. Structural response

The structural response is presented in the form of the distribution of maximum inelastic strain along the normal section through the wall. The distribution of the inelastic strain is shown for different time steps throughout the analyses. The definition of inelastic strain for the damage-plasticity model is shown in Eq. (9). For the damage model, the plastic strain is excluded and for the plasticity model, the damage variable is zero.

$$\varepsilon_i = \varepsilon_p + \omega\left(\varepsilon - \varepsilon_p\right) \tag{9}$$

where ε is the inelastic strain, ε_p the plastic strain and ω the damage variable. The time steps for which a new distribution of the crack propagation is presented were chosen as the time during which a compression wave travels from the left side of the wall to the right side and then back to the left side as a release wave in undamaged concrete; thus, $\Delta t = 2L/U_s$ where $U_s = \sqrt{E/\rho}$ is the elastic wave speed in undamaged concrete. The total number of strain distributions presented is 10; thus, the last distribution will be $t = 1.68$ ms.

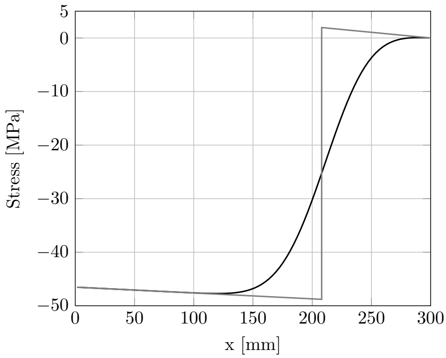


Figure 6: Stress distribution at the time of crack initiation for analytical solution. Stress distribution in numerical analysis (black) and analytical solution with jump condition (gray)

When the brittle crack spalling approach is used to determine the position of the first spalling crack, the following equation can be used for the linear decrease of a plane pressure wave, [14]:

$$x = \frac{U_s \cdot f_{ct}}{2 \cdot \left| \dfrac{\partial p}{\partial t} \right|} = 92mm \tag{10}$$

where U_s is the elastic wave speed in undamaged concrete and $\frac{\partial p}{\partial t}$ the pressure-time gradient of the load.

By using the brittle crack spalling approach, the linear decrease of pressure would result in additional spalling cracks which appear at a distance of 92 mm from the position of the previous crack. This will continue until the release wave reaches the loaded side of the wall.

In the numerical models, the first position of the crack initiation appeared at a distance of 97 mm from the right side of the wall. The deviation from the brittle crack spalling approach was attributed to the stress-time singularity of the pressure wave which could not be represented without a jump condition (Fig. 6). The final distribution of inelastic strains from the numerical models along the section through the wall, as well as intermediate distributions throughout the analyses, are presented in Fig. 7. The position of the spalling crack was interpreted as the middle of the strain localisation zone. For the damage model, where individual damage parameters where used in tension and compression, no fully developed spalling crack in the analysis occurred. However, the damage distribution seems to develop in a manner similar of the plasticity model. The position of the spalling cracks for the various approaches are presented in Table 3.

The main difference between the smeared crack band approach and the brittle crack spalling approach was the timing of the crack propagation. Where, according to brittle crack spalling, a spalling crack was supposed to appear momentary at the time of crack initiation only a fraction of crack propagation was achieved during the first wave cycle in

Table 3: Position of spalling crack in the different analyses

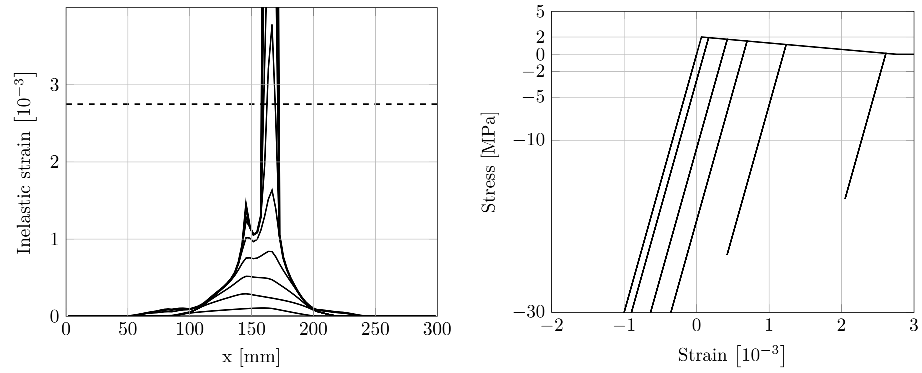
Approach	Crack initiation	Spalling crack
Brittle crack spalling	92 mm	92 mm
Strain-softening crack spalling		
Plasticity model	97 mm	140 mm
Damage model	97 mm	No fully developed spalling crack
Damage-Plasticity model	97 mm	92 mm (fully developed) 150 mm (Partly developed)

the numerical models. After the initial wave cycle, where a compression wave moves from the left to the right side and back to the left side as a release wave, only a small amount of inelastic strains were achieved within the cracked region. This result was expected since the tensile strength was included in the numerical approach compared to the brittle crack spalling approach in which the tensile strength was only a measurement to determine where the spalling cracks were to appear.

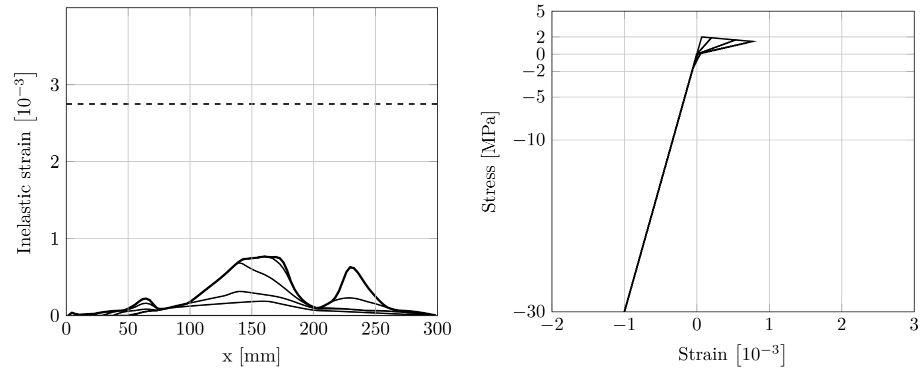
Fig. 7 also shows the stress-strain relation throughout the entire analysis at the position where strain localization occurred. For the damage model, the position was chosen at the point where the highest inelastic strains were achieved. Importantly, for the different material, models the development of the inelastic strain took place during a non-monotonic material response where the inelastic strain increased during the tensile response of each cycle. For each cycle, the inelastic strains increased until a fully open crack was achieved. This response contradicts the hypothesis that a fully opened crack appears momentarily when the tensile strength is exceeded by the first release wave.

4.3. Spalling velocity

The velocity of the section of the concrete wall which is separated from the rest of the wall by a fully opened crack is defined as the spalling velocity. The spalling velocity is the speed with which the section spalled off will be released at the unloaded side of the wall. In [14], this velocity is defined as the particle velocity at the moment of crack initiation. Two main differences compared to the brittle crack spalling approach are achieved in the numerical models. First, the moment of crack initiation is not where a fully opened crack is formed. Second, since the wave length of the loading pressure is much longer than the length of the wall, a pressure is still applied on the loaded side of the wall during the phase of crack propagation. Because of this fact, the part which will eventually be spalled away from the rest of the wall will be pushed in front of the remaining wall section, increasing the particle



(a)



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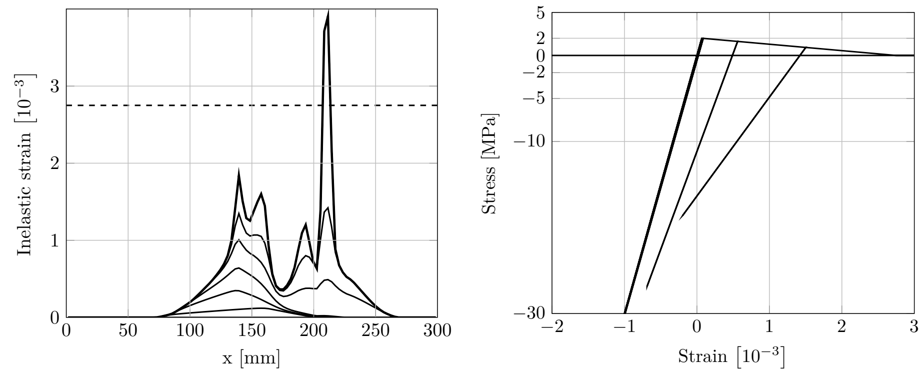


(c)

Figure 7: Response based on the finite element analysis showing the distribution of inelastic strains through the wall at each time step Δt (left) and stress-strain response for the element with highest inelastic strain (right), (a) for the plasticity model, (b) for the damage model and (c) for the damage-plasticity model.

velocity for each load cycle. The effect is that the final distribution of particle velocity along the wall is constant, when the loading pressure has reached zero. If a difference between the spalled off part of the wall and the retained part appears it would be related to the final wave cycle. The particle velocity for the retained left side of the wall and the spalling velocity at the right side of the wall for the different analyses are presented in Fig. 8 along with an analysis with linear elastic material response of the concrete for comparison.

4.4. Element mesh dependency

A sensitivity analysis of the mesh density was performed. The distribution of inelastic strains were compared between a coarse mesh with element size 6 mm and the total number of elements $N = 50$, and a fine mesh with element size 1.5 mm and the total number of elements $N = 200$. In Fig. 9, the non-monotonic material response during crack propagation for the element featuring the highest inelastic strain is presented. All mesh sizes experience non-monotonic response during crack propagation. The number of wave cycles, Δt, seems to decrease with an increased number of elements. But since the distribution of the strain localisation also seems to decrease with an increasing number of elements, the procedure of choosing the fracture zone at three times the maximum aggregate size should be adjusted to yield a less brittle material. Thus, a slower development of the inelastic strains during the non-monotonic response. The conclusion of the mesh dependency is that the general response which shows that the crack propagation takes place during the non-monotonic material response is true for arbitrary mesh densities.

5. Discussion

5.1. Overview

When the tensile strength of a material and the material softening was taken into account, the damage propagation which leads to spalling will be different compared to an approach in which a crack is considered fully developed as soon as the tensile strength is reached. According to the numerical analysis presented, when tensile strength and softening were included, spalling occurred after cyclic development of the inelastic strain. The load duration, which influenced the pressure-time gradient, affected the number of cycles needed to develop the final strain localisation.

5.2. Structural response

The evaluation of the structural response focused on the distribution of inelastic strains in order to estimate positions of cracks and crack development across the wall studied. Different constitutive models were used to describe the softening and, in particular, to describe the response during repeated loading and unloading. From the distribution of inelastic strains, it becomes clear that the material response during repeated loading and unloading is of great importance when crack propagation resulting in spalling takes place.

Since the material responses before crack initiation are the same, the first crack initiation takes place in the same section for the various material models. However, since the material response during cyclic crack propagation differs between the models, the response and final

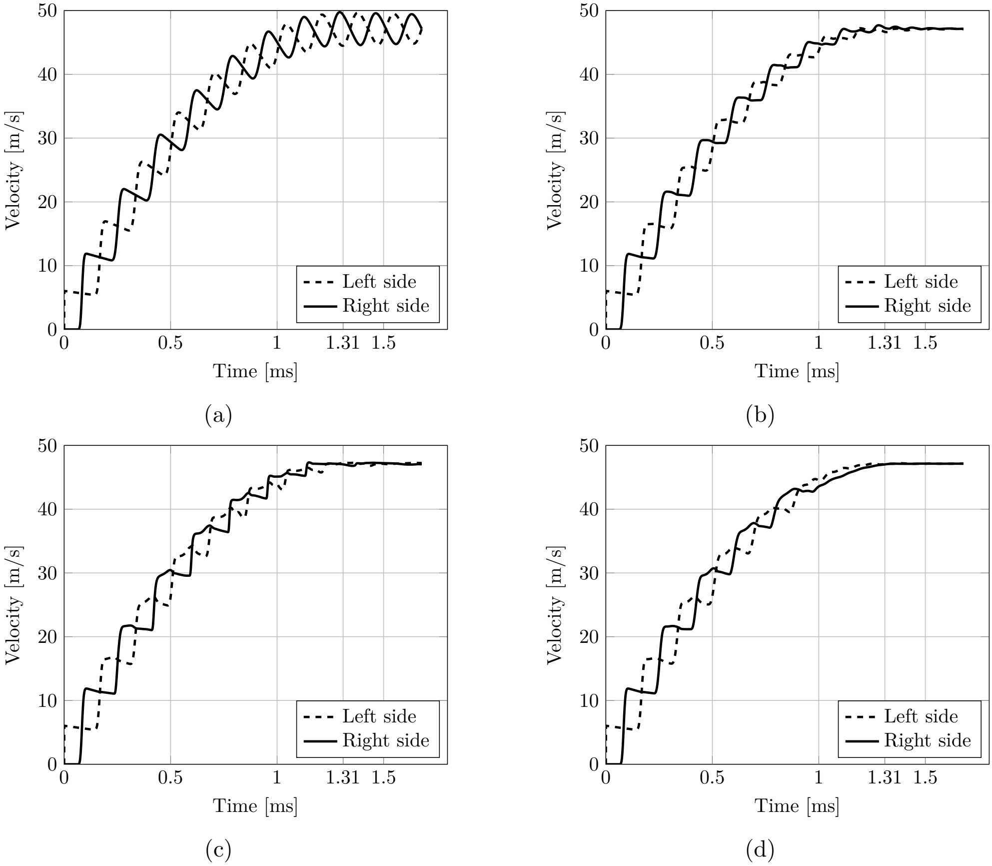
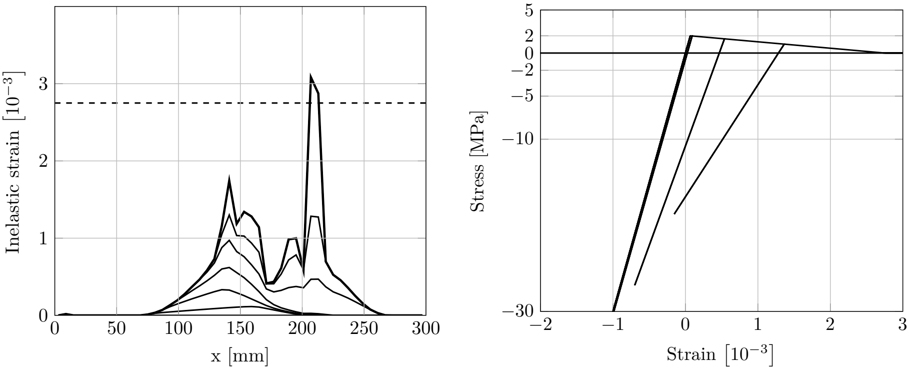
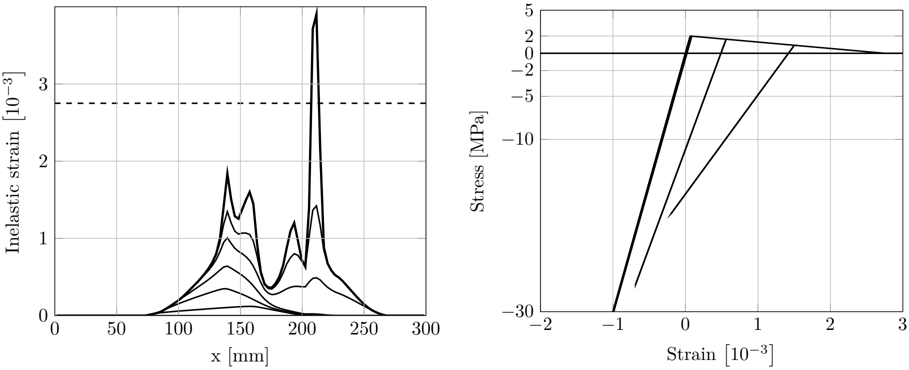


Figure 8: Particle velocity during the analyses at the retained left side (loaded side) and the spalled right side of the wall for (a) elastic material response, (b) plasticity model, (c) damage model and (d) damage-plasticity model.



(a)



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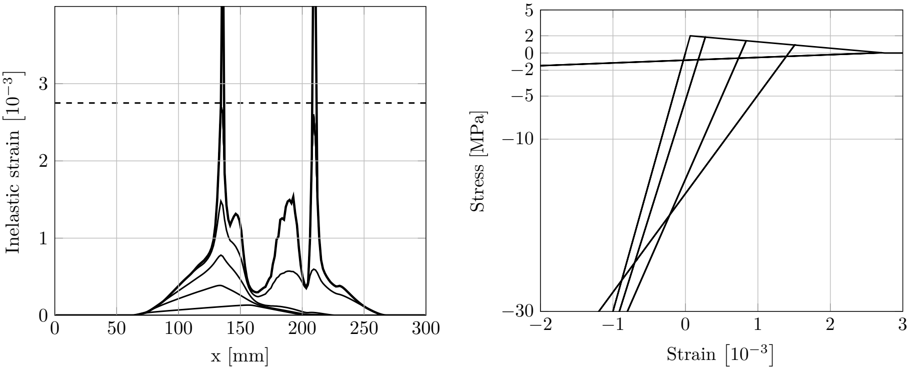

(c)

Figure 9: Mesh size sensitivity showing the distribution of inelastic strains for the damage-plasticity model through the wall at each time step Δt (left) and stress-strain response for the element with the highest inelastic strain (right), (a) coarse mesh $N = 50$, (b) original mesh $N = 100$ and (c) fine mesh $N = 200$.

crack distribution resulting in spalling damage will be different. Since the wave speed along the structure will vary due to partial crack development, the stiffness distribution in the wall affects the forthcoming wave propagation. The similarity between the analyses is that the initial spalling crack does not develop in the same section as predicted by the simplified brittle crack spalling approach. The reason is that in order to develop a discontinuity in the deformation along the wall, i.e. a crack, a relative difference in particle velocity between opposite sides of a material point is needed over a time span. This time span is too brief in the performed analyses. Thus, to achieve an instant discontinuity in the deformation along the wall when the first release wave arrives, a crack must be fully developed already. Since the tensile strength and the tensile softening are included in the numerical analyses, the inelastic strain which can be developed in this section will be infinitesimal and therefore the crack propagation will be at most limited. In the analysis of the damage-plasticity model a secondary crack propagation takes place before the first crack has fully developed. This second crack seems to appear at the exact position where the brittle crack spalling approach predicts the first crack to appear. No parameter studies with regard to geometry, load intensity, load duration, load shape, material strength and shape of softening branch have been performed. Thus, no conclusions can be drawn as to whether this second crack is one of the final spalling cracks in the general case. However, it is clear that the propagation of spalling cracks is fundamentally different between the two approaches.

5.3. Local crack propagation

All analyses show that if a spalling crack were initiated, it would need a certain time to fully open. In the numerical analyses the length of the stress wave was longer than the length of the concrete wall studied. Thereby, the material points at the final crack were subjected to repeated compression and tension waves. This loading condition resulted in a crack opening process that was driven by the tension waves. In the pure damage model, a fully open crack was not created. However, a fracture mode of the structure similar to that due to spalling might still appear depending on the boundary conditions and response for the entire wall structure. This damage would in that case be a secondary spalling damage.

5.4. Spalling velocity

The final velocity distribution throughout all the analyses were close to constant. Thus, no major separation between the different parts of the wall took place during or at the end of each analysis. In order to achieve a rapid separation, there has to be a velocity gradient somewhere in the structure. However, due to the spalling damage, some part or parts of the wall might no longer be attached to the supports. Thus, a velocity gradient can be created at a later stage of the global structural response.

5.5. Motivation to new hypothesis

By studying the previous approach, brittle crack spalling, to determine spalling damage and the effect due to spalling, it becomes clear that this approach cannot properly reflect the response of a structure consisting of concrete material. In order to evaluate the effects of different concrete materials when studying spalling an understanding of the material as well as the structural response is required. The previous approach to describe spalling predicted, for example, the response of fibre reinforced concrete to be exactly the same as for plain concrete or any brittle material as long as the tensile strength and material stiffness were the same. The new hypothesis predicts that the post crack behaviour of the material influences the response. Thus, the response of fibre reinforced concrete and plain concrete is no longer by definition the same. The numerical analyses also show that, for a pressure wave where the wave length in concrete is much greater than the thickness of the structure, crack propagation could be highly influenced by the response of the material during cyclic development of inelastic strain. The cyclic development of crack propagation which may occur for spalling damage is a feature which the previous approach to describe spalling damage failed to acknowledge.

6. Conclusions

Based on the numerical study performed, it can be concluded that a spalling crack from a pressure wave does not occur instantaneously nor does it need to occur at the position where the tensile strength is first reached. The time frame within which to reach inelastic strain which corresponds to a fully opened crack can be long enough for the critical section to undergo non-monotonic response. For each new cycle, the inelastic strain increases until a fully opened crack finally can appear. The study also shows that the material model chosen with respect to the response for cyclic loading is important for the development of inelastic strains, in addition to the final position for the spalling crack.

7. Further studies

In this study, only the linear decrease of the pressure wave and linear crack-softening of the material have been studied. In reality, both the pressure decrease and crack-softening of concrete seem to correspond to an exponential decrease. Further studies should include these properties of the load and their material response. Bi-linear softening, which is usually considered an acceptable simplification to the response of concrete, should be investigated and compared to both linear crack-softening and exponential crack-softening. It is also important to include a study with variations of geometry, i.e. wall thickness; material properties, such as tensile strength and fracture energy; and variation of load intensity, such as peak pressure and load duration.

Acknowledgements

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Paper B

Finite Element Analyses of Concrete Structures Subjected to Blast Loads with a Damage-Plasticity based Material Model, CDPM2

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To be submitted for international journal.

Finite Element Analyses of Concrete Structures Subjected to Blast Loads with a Damage-Plasticity Based Material Model, CDPM2

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Abstract

This paper studies the response of reinforced concrete structures subjected to blast loads. Numerical models are used to evaluate the numerical response of a simply supported reinforced concrete beam and a one-way supported slab with a combined damage and plasticity constitutive model for concrete, CDPM2. Previous research has shown that strain-rate dependent material parameters might be overestimated for higher strain rate. In this study, these features are evaluated for reinforced concrete structures where bending is the dominating response. The numerical analyses indicate that fracture energy during tensile fracture and how this value is chosen have larger effects on the deformations of the structure than whether or not the strain-rate dependency of the material properties are taken into account. It is also concluded that mesh size and modelling techniques may have a large impact on the resulting response of the structure in the numerical analysis.

Keywords: concrete, blast load, numerical modelling

1. Introduction

1.1. Background

The interest for building safety with regards to highly dynamic events such as close by explosion has accelerated after the recent decades of terrorist attacks. The interest has expanded from civil defence shelters and pure military targets to also include civil buildings used for different civil functions. Due to the rather rapid shift of area of interest, the general engineering community lack much of the knowledge and tools with which to design and evaluate structures with respect to dynamic events with very fast transient and high magnitude of peak loads.

One of the most common building techniques to withstand highly dynamic events is reinforced concrete. To describe the material response of concrete, many different constitutive models exists. In order to better represent the material response of concrete for highly

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dynamic events, such as blast- and fragment loading, a combined damage and plasticity model has been developed outside of this project, that also takes into account the increased material strength of high strain-rates, CDPM2 [1, 2].

1.2. Problem identification

CDPM2 has previously been validated by simulations of material tests for high multiaxial stress states, [1, 2]. However, no simulations where CDPM2 successfully is used for a reinforced concrete structure have been performed.

1.3. Aim and objectives

The aim of this study has been to investigate the dynamic response of numerical models of reinforced concrete structures subjected to blast loads using the material model CDPM2 as it is implemented in LS-DYNA [3]. The objectives have been to study deformations and failure modes of reinforced concrete beams and slabs. Another objective has been to evaluate the effects of previously identified limitations in CDPM2 when modelling structural elements subjected to highly dynamic events, [2]. This was approached by varying the initial value of the fracture energy for tensile failure of the concrete and by studying the effect of increased material strength due to high strain-rates.

1.4. Method

A literature study was conducted to find suitable experimental studies which could be used to compare against numerical models. An appropriate level of detail of the numerical models was chosen in order to capture the deformations and failure modes of the structures. From this procedure, the structural response of the chosen experiments was studied in finite element models with the latest implementation of CDPM2, [1, 2]. The structures studied have previously been tested in different experiment series; a study of blast loaded beams [4] and a blind simulation competition of blast loaded slabs [5, 6, 7]. Both experimental series studied bending as the main structural response.

The modelling techniques were based on general approaches for finite element models for highly dynamic events. The results of the experiments with respect to maximum deformations and failure modes were compared to the results from the numerical models.

1.5. Limitations

In this, study only reinforced concrete beams and slabs subjected to blast loads were studied. The main influencing response was bending and to some extent shear. The study did not include an evaluation of different types of impacts nor penetrations from such items as fragments or projectiles where high hydrostatic pressures are achieved. The study did not focus on influences of modelling techniques even though some issues regarding modelling techniques have been treated. The numerical analyses have only been carried out in one finite element code, LS-DYNA [3].

2. Material properties and material models

2.1. Concrete properties and applied model

Concrete structures subjected to highly dynamic events are to a large extent subjected to confinement due to different inertia effects compared to static load conditions. This creates demands on the constitutive models used in numerical analysis in order to take these effects into account. In general, constitutive models for high confinement pressure focus on the response in compression and how peak pressures and stresses are achieved. For structures subjected to static loads, the peak load and deformation abilities are usually of interest. However, for structures subjected to large dynamic loads, other aspects of the response are of greater interest, such as the maximum deformation or the extent of the damage to the structure. The damage could be estimated through the amount of concrete breaking loose from the structure or the penetration depth of an object. These demands allow the structure to deform far beyond the point where the peak stress in the materials emerge. Thus, the ability to describe the response of the materials during fracture becomes important.

CDPM2 has been developed as a material model which intends to describe the response of concrete to regular stress conditions, as well as high triaxial stress states. This include pre-peak hardening, crack-softening response and non-linear compaction, as well as the influence of a high strain-rate such as increased capacity in tension and compression. The specification of requirements for dynamic load conditions was specified in [2]. The first implementation of the material model in a finite element code was carried out in OOFEM [8, 9]. The additions with regard to dynamic response [2] has been implemented in LS-DYNA [3], which is evaluated in this study.

2.2. Material response of concrete during high strain-rate

When concrete is subjected to a high strain-rate, i.e., increased deformation in short time frames, the material characteristics change compared to low deformation rates. The most distinct change in material properties is the increased strength, both in compression [10] and in tension [11].

The increased tensile capacity is explained as a combination of increased strength due to the bridging of stresses over water-filled pores in the concrete [12] and a change of fracture mode where a crack, caused by increased tensile stresses during high strain-rates, tends to go through the aggregates in the concrete rather than around them as happens during lower strain-rates [13]. Increased tensile strength of up to about 7 times the strength during static conditions has been observed for very high strain rates [11].

For compression, the increased strength during high strain rates is not as high as for tensile loads. Increased compression strain rates can lead to an increase of material strength of up to 2,25 times the capacity during static load conditions [10]. In compression, however, the influence of mass inertia is larger. During axial compression of an experiment specimen to determine the compression capacity, the transverse expansion of the specimen is prevented due to mass inertia. This result leads to a natural confinement of the material inside the specimen which sometimes has been interpreted as an increase in uniaxial compression capacity.

In [2] it has been shown through 3D continuum finite element analysis that a natural structural confinement effect appears for high strain rates resulting in increased compression resistance for the specimen without any constitutive laws describing a strain rate dependent material strength. In general, the increased compression capacity is implemented as increased strength for higher strain-rates in the constitutive models. In [2] it is shown that this approach results in an overestimation of the increased compression capacity for higher strain rates when used in 3D continuum analyses.

The material ductility of concrete is represented by the fracture energy. In a smeared crack model, this is modelled as a strain softening branch in the stress-strain relation. If the fracture energy is kept constant, an increase of the tensile capacity will lead to increased brittleness. The number of studies where the fracture energy has been studied with regards to its dependency of strain-rates are limited. In [14, 15] it is shown that the overall effect of increased strain rates is increased fracture energy in a similar way as the tensile capacity is affected by increased strain rates.

In CDPM2 the softening branch of the stress-strain relation in tension is treated by using the dynamically increased tensile strength as the new tensile capacity. For bi-linear softening, the softening develops with the same stress-strain gradient, $\delta\sigma/\delta\varepsilon$, as for the original tensile strength until a third of the dynamically increased tensile strength is reached. From this point the softening branch has the same gradient as the unaffected softening branch until zero stress is reached (see Figure 1).

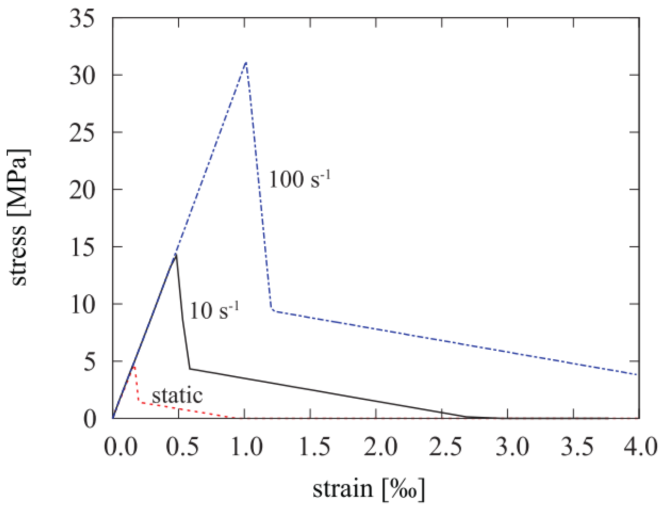


Figure 1: Softening branch for CDPM2 with dynamically increased tensile strength for different strain-rates taken from [2]

3. Experiments studied

3.1. Overview of studies

To investigate the performance during dynamic response of high intensity load conditions, two different experiment series were studied, [4] and [5, 6, 7]. One of the experiments chosen was a reinforced concrete beam and the other a reinforced concrete slab , both loaded with blast loads and one-way supported. Normal strength concrete and conventional steel reinforcement were used in both cases. One study was chosen to represent a moderately dynamic response due to blast loads with respect to deformations of the structure, [4], whereas the second study represented an upper limit with regard to deformations without a complete collapse of the structure [5, 6, 7].

3.2. FOI beam

The Swedish Defence Research Agency (FOI), has over the years conducted experimental studies of reinforced concrete beams with various concrete strengths [16, 17, 18]. From the results of these experiments, a numerical study was conducted to investigate the capability of finite element models [4]. In [4] the RHT-model [19], was used to describe the material response of concrete. In this study, the CDPM2-model was used to describe the material response of the concrete. However, the same basis for assessing a numerical model as in [4] was used to study the response of a blast loaded reinforced concrete beam.

The beams in [4] are considered to have a rather normal dynamic response for a beam or slab-like structure subjected to detonation when it comes to deformations during the structural response and the design of the beam. The mean concrete compression strength was given to be $f_{cm} = 50$ MPa and the reinforcement was specified to be of quality B500B [4]. The span between the supports was 1500 mm. The hight of the beam was 160 mm and the width was 300 mm. The beam was reinforced by $5\phi16$ as longitudinal reinforcement and stirrups $\phi8$ $s200$ (see Figure 2). Detailed description of the beam can be found in [4].

3.3. Blind simulation blast test

In 2012, a blind simulation was carried out as a competition where the participants were asked to predict maximum deformation and an over-all response of four different slabs that were later tested. The slabs were subjected to blast loads generated by a Blast Load Simulator (BLS). One participant, [6], has presented results of simulations where the performance of a number of constitutive models was compared.

The dimensions of the slabs were specified as $6''\cdot33.75''\cdot4''$ (1626 mm·857 mm·101.6 mm); see Figure 3. Normal strength concrete, high strength concrete, conventional steel reinforcement and high strength steel reinforcement were combined into the four different set-ups. Only the slab with normal strength concrete and conventional reinforcement was studied here. The slab was supported along the shorter sides by a two piece steel frame assembled by steel tubes. The support frame, see Figure 4, is briefly described in [5] and [6].

The concrete strength of the slab in the experiment was of normal strength, ($f_{cm} =$ 34.5 MPa), and the reinforcement had a yield strength of about $f_y = 480$ MPa. The reinforcement was arranged in two directions. According to the input to the blind simulation

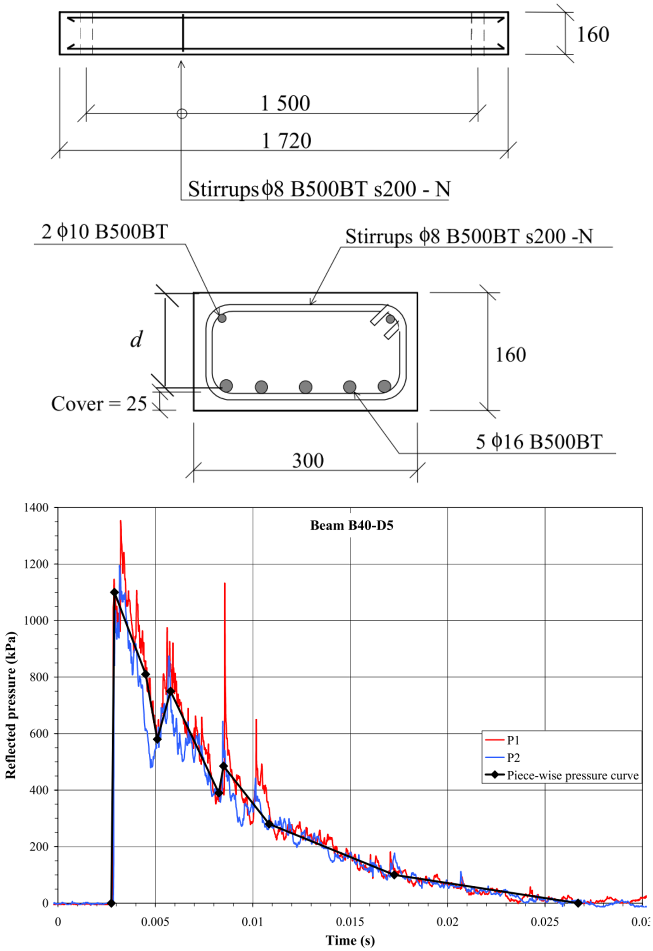


Figure 2: Geometry and reinforcement layout (above) and measured pressure over the beam and simplified load used in the analyses (below); both pictures taken from [4]

competition, the slab was only reinforced in the bottom, i.e., no compression reinforcement at the top of the slab [5, 6].

The load was generated by the BLS and described as pressure-time histories in the input to the Blind simulation blast test. Two different pressure-time histories were provided. The first case resulted in an impulse of 7.04 MPa · ms and the second gave an impulse of 5.38 MPa · ms [6]. In the analyses performed for this article, the higher impulse from the Blind simulation blast test was used; see Figure 5.

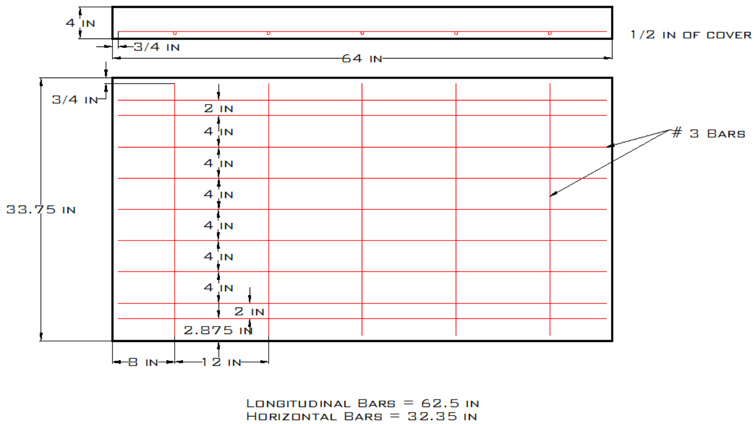


Figure 3: Reinforcement arrangement and geometry of Blind simulation slab taken from competition input; also presented in [6] and [5]

Figure 4: Mounting of Blind simulation slab with steel frame taken from competition input; also presented in [5]

The response to the Blind simulation blast test was more extreme when it came to the deformations during the response compared to the FOI beam [4]. To determine the bending deformation of a structure, a commonly used measurement is the ratio between the maximum deformation and the length of the member , δ/L, [20]. For the beam in [4], this ratio is approximately $\frac{1}{60}$ which can be considered as a response where the normal assumptions for an evaluation of a beam holds. In the Blind simulation blast test, the maximum deformation over length ratio was approximately $\frac{1}{13}$. [20] categorises $\delta/L = \frac{1}{15}$ as very large deformations where special reinforcement arrangements should be made in order to, in a reliable way, provide structural resistance and integrity.

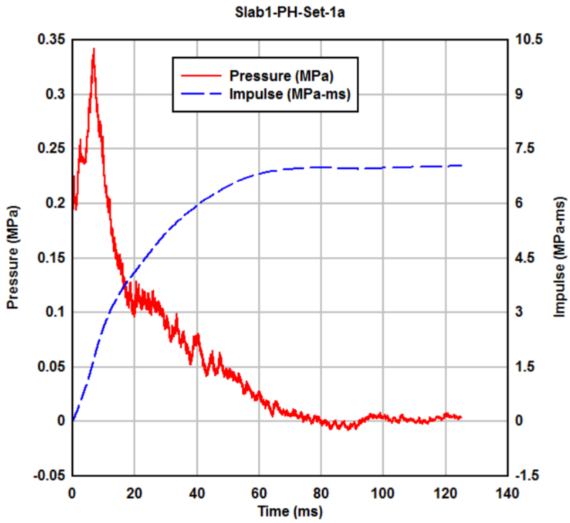


Figure 5: Measured pressure for blast load in Blind simulation blast test taken from competition input; also presented in [6] and [5]

4. Finite element models

4.1. Overview

Both experiments were modelled using a similar approach. The analyses were carried out in LS-DYNA with 3D finite element models. The concrete was modelled with 3D continuum elements whereas the reinforcement was modelled with beam elements. Support conditions were modelled with 3D continuum elements for the FOI beam in a similar way as described in [4] whereas shell elements were used for the Blind simulation blast test [6, 5] and perfect bond was assumed between the reinforcement and surrounding concrete. The concrete model is based on isotropic damage; thus, no direction of a crack is stored in the analyses. Instead, the crack direction will be perpendicular to the largest principal strain.

4.2. FOI beam

4.2.1. Material input

The reinforced concrete beam was analysed by [4], but using the RHT-model, [19]. The geometry of the beam, reinforcement amount and support conditions were applied as in [4]. However, some information is limited. Therefore, parameters have been adopted from [4] and computed according to Model Code 2010 [21]; see Table 1. When numerical studies are performed, the chosen fracture energy and the shape of the strain-softening branch influence the response and crack pattern in the structure. In general, the energy consumed during the fracture of the beam tends to be overestimated since cracks appear too close to each other [22]. In the analyses performed, it was found that in the area close to the support, extensive shear cracking appeared with many adjacent elements cracked. Therefore, the structural response for both the fracture energy acquired from [21], 148 Nm/m^2, and half of that value, 74 Nm/m^2, was studied. Bi-linear tensile softening was chosen for all analyses with stress-crack opening response according to [23]. In compression, exponential softening was used after the compression strength had been reached [1]. The crack band width [24] was based on the length of one element; thus, the localisation zone of one crack was supposed to be extended over one element row.

Table 1: Concrete material parameters FOI beam

Material parameters FOI beam		
		According to
Module of elasticity	36.8 GPa	[21] based on f_{cm} from [4]
Compression strength	50 MPa	[4]
Tensile strength	3.6 MPa	[21] based on f_{cm} from [4]
Poison's ratio	0.2	[21]
Density	2350 kg/m^3	
Fracture energy	148 Nm/m^2 74 Nm/m^2	[21] based on f_{cm} from [4]

Material input for the reinforcement was taken from [4]; see Figure 6. The modulus of elasticity was chosen to 200 GPa, Poison's ratio to 0.3 and the density was set to 7800 kg/m^3. A piecewise linear stress-strain relation was used for the steel material.

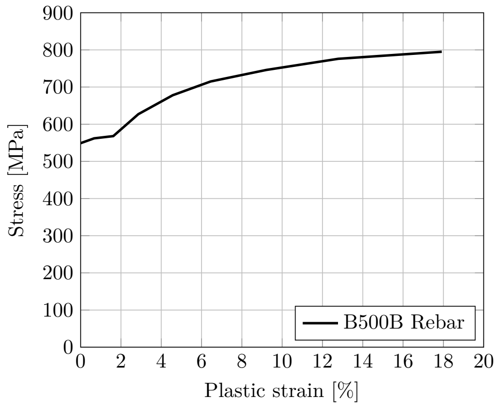


Figure 6: Stress-strain response of reinforcement for FOI beam adapted from [4]

4.2.2. Elements and mesh

The concrete in the beam was modelled with 3D hexahedral elements. The reinforcement in the beam was modelled with beam elements. The main reason was the limitation in choice of material models in LS-DYNA for truss elements. A more detailed review of how reinforcement can be modelled in LS-DYNA is presented in [7]. The support roller and the concrete were modelled with solid hexahedral elements. A mesh size of approximately 10 mm, as in [4] was originally used for the concrete elements. The mesh size was later divided by two since it was found that the shear cracks occurring in the analysis was better represented by using smaller element size.

The connection between concrete and reinforcement was performed by using constraints between the concrete and reinforcement. By using the *CONSTRAINED_LAGRANGE_IN_SOLID function in LS-DYNA, the concrete and reinforcement elements did not need to be modelled with shared nodes in order to to be connected [7]. The contact between the support roller and the concrete beam was treated with full interaction between the members.

To limit the computation time for the analyses, the beam was modelled with symmetry faces along the middle of the beam and at the middle of the span. By these symmetry faces the number of elements in the model was reduced to one-fourth compared to the case when the whole beam would have been modelled.

4.3. Blind simulation blast test

4.3.1. Material input

The material parameters of the concrete slab were taken from the experiment description for the Blind simulation blast test which are documented in [6, 7, 5]. The available material data which were specific to the used concrete was limited to the uniaxial compression strength. Tensile strength together with modulus of elasticity was for instance not presented. In general, the crack softening behaviour of concrete is considered to be just as important for the the results of a numerical model [22]. For all material properties not presented in the competition documentation, values were chosen in accordance to Model Code 2010 [21]; see Table 2.

To study the effect of the fracture energy, two different fracture energies were used, 138Nm/m^2 which corresponds to the value given by Model Code 2010 [21] and 69Nm/m^2 which is half of this value. As for the FOI beam described in Section 4.2.1, bi-linear softening was used in tension and exponential softening was used in compression.

Table 2: Concrete material parameters FOI beam

Material parameters FOI beam		
		According to
Module of elasticity	32.5 GPa	[21] based on f_{cm} from [6]
Compression strength	34.5 MPa	[6]
Tensile strength	2.7 MPa	[21] based on f_{cm} from [6]
Poison's ratio	0.2	[21]
Density	2350 kg/m^3	[6]
Fracture energy	138 Nm/m^2 69 Nm/m^2	[21] based on f_{cm} from [6]

Material input for the reinforcement was given before the blind simulation competition as stress-strain response; see Figure 7. The modulus of elasticity was chosen to 200 GPa, Poison's ratio to 0.3 and the density was set to 7800kg/m^3.

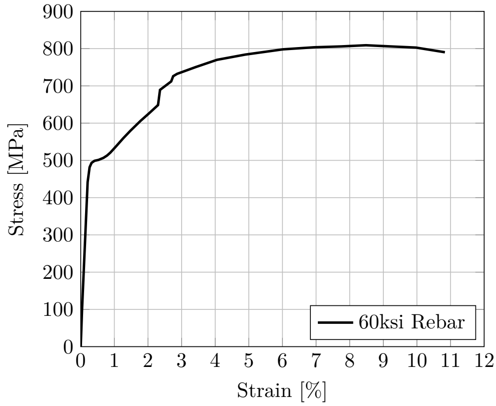


Figure 7: Stress-strain response of reinforcement adapted from competiotion input and [6]

4.3.2. Elements and mesh

The reinforcement was modelled in the same way as for the FOI beam with beam elements and a piecewise linear stress-strain relation; see Section 4.2.2. The mounting frame assembled from steel tubes which composed the supports of the slab was modelled with shell elements. The concrete slab was modelled with 3D continuum hexahedral elements. An original mesh size of approximately 17 mm in the slab thickness direction was determined to be sufficient in [7]. This mesh size was later divided by two to avoid unrealistic de-lamination of the concrete along the reinforcement in the FE-analysis which might lead to incorrect deformation shape for the slab.

The current implementation of CDPM2 did not allow mesh dependent input for the stress-strain response in compression [1]. Instead, the response was based on a pre-defined fracture energy of 2200 Nm/m^2 and an element size of 100 mm. This could lead to a more brittle response than desired if smaller elements were used. In the analyses, different convergence issues in the material model appeared at the bottom of the slab close to the support at the compressed side and at the top of the slab in the middle of the span. It was believed that one of the problems might be the brittle response that was achieved due to the small elements. Therefore, the outer layer of concrete elements was replaced in these areas with linear elastic elements; see Figure 8.

The coupling of concrete and reinforcement was done in the same way as described in Section 4.2.2. The contact between the support structure and the slab was modelled with 2D surface contact without any friction coefficient.

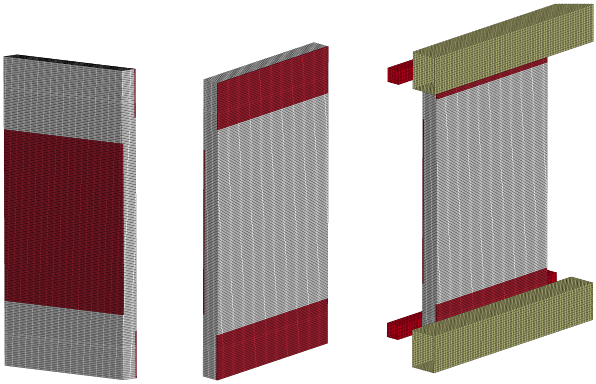


Figure 8: Elastic elements in the numerical model of the Blind simulation slab. Top sid (left), bottom side with and without support structure (middle and right)

5. Results of modelled experiments

5.1. Overview of results

The results of the numerical analyses are presented as deflection histories at the point on the tensile side of the mid-section of the beam and the slab. The numerical results are verified by the deformation modes and the maximum deformations. Furthermore, the type of failures which dominates the structures are of great importance.

5.2. FOI beam

The deformations from the analyses and the experiment from [4] of the FOI beam are presented in Figure 9, ranging from 36-67 mm. All analyses overestimated the deformation of the beam in comparison with the experiment (24 mm).

The crack pattern for the beam test [4] is compared to the contour plot of the tensile damage from the analyses in Figure 10. From Figure 10 it becomes clear that a much larger extent of shear cracking takes place in the numerical analyses than in the experiment.

5.3. Blind simulation blast test

The deformation histories for two of the slabs were taken from [7] and [5] and are presented together with the results from the numerical analyses in Figure 11. A rather large difference in maximum deformation at the mid-span of the slab is achieved for the numerical models where variation of the fracture energy and the strain rate dependency for the concrete strength are investigated. The tests of the slabs had a maximum deformation of 108 mm and 113 mm. Three tests were conducted but one of the slabs was subjected to the lower pressure-impulse load [5]. The other two are presented in [5] and Figure 11. The numerical analyses show a variation in the maximum deformation of between 78 mm and 152 mm. With respect to maximum deformation, the analysis with fracture energy of 138 $\mathrm{Nm/m^2}$

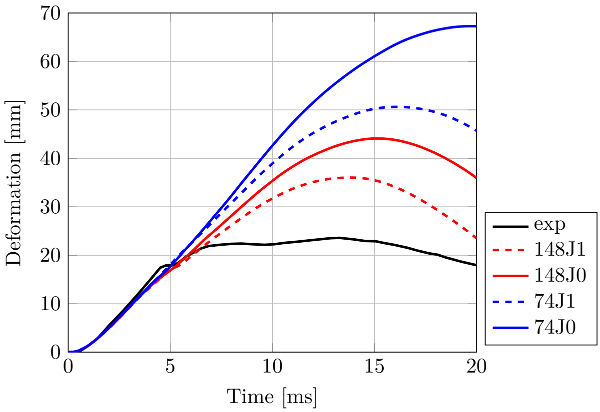


Figure 9: Maximum deformation vs time history for the analyses of the FOI beam for different fracture energies, 148 and 74 Nm/m^2, and strain rate dependent strength turned on, 1, and off, 0.

without strain rate dependent concrete strength and the analysis with fracture energy of 69 Nm/m^2 with strain rate dependent concrete strength both show results very close to the test result from the blind simulation competition, 102 mm and 125 mm, respectively.

Due to very large deformations of the slabs, the damage of the tested slabs is very severe which is also seen in the numerical models. The slabs are at the point of maximum deformation fully damaged and individual cracks can no longer be distinguished; see Figure 12.

5.4. Element mesh dependency

The element mesh dependency was evaluated based on the mechanisms of the models and the level of detail of the actual failure mode. The mesh density which was determined in [6] for the Blind simulation blast test was refined in order to reduce the effects of delamination of the concrete at the level of the main bending reinforcement close to the support. The delamination caused a failure mode where two yield lines appeared across the slab; see Figure 13. Since the available photo for the Blind simulation blast test, also in Figure 13, indicated that only one yield line appeared, the mesh was refined until the same response was achieved in the finite element models.

In [25] it was concluded that an element size of 12 mm was sufficient to capture the bending behaviour for a blast loaded beam similar to the FOI beam. However, the same mesh size as in [4], 10 mm, was originally also used in the analysis in this article. But, to better reflect the shear crack that occurred in the analysis, it was concluded that a finer mesh better represented the response; see Figure 14. The element sizes used for the concrete in the two cases, beam and slab, are presented in Table 3.

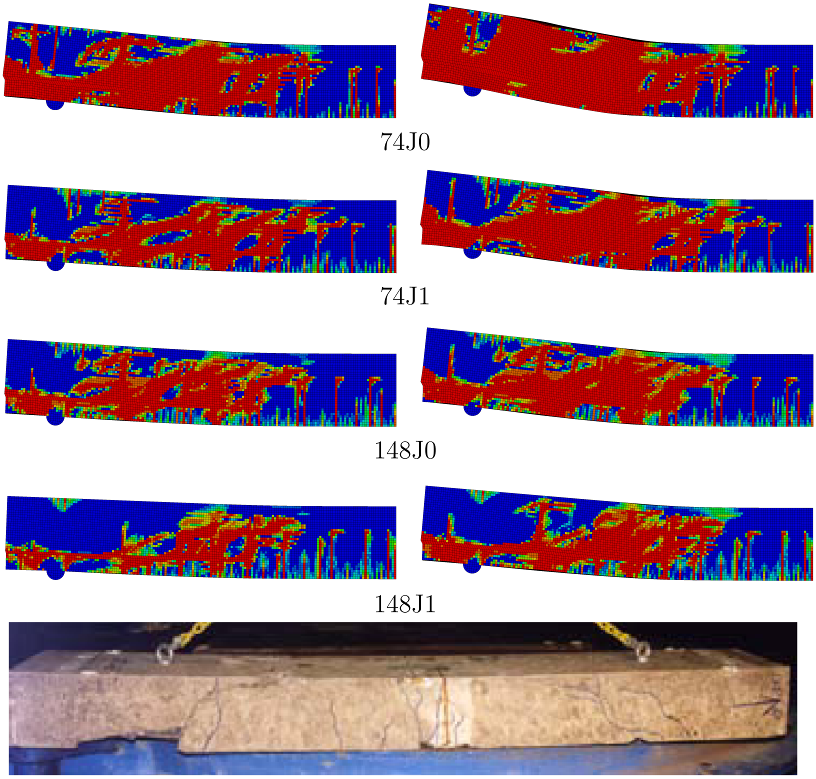


Figure 10: Contour plot of damage and the crack pattern of the tested beam (taken from [4]) for the FOI beam for the different fracture energies, 148 and 74 Nm/m^2, and strain rate dependent strength turned on, 1, and off, 0. Left contour plots at the time of half maximum deformation and right plots at maximum deformation. Red colour represent full damage and blue represent no damage.

Table 3: Mesh sizes used in the

Mesh sizes			
Study	Original mesh	Refined mesh	Cause for refinement
FOI beam	10 mm	5 mm	Detail level of shear failure
Blind simulation slab	17 mm	8.5 mm	Failure mode in bending

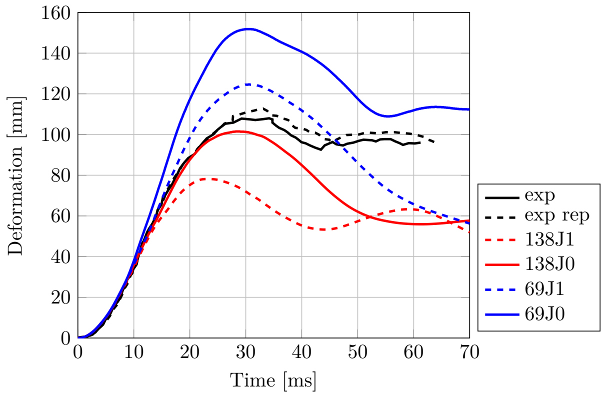
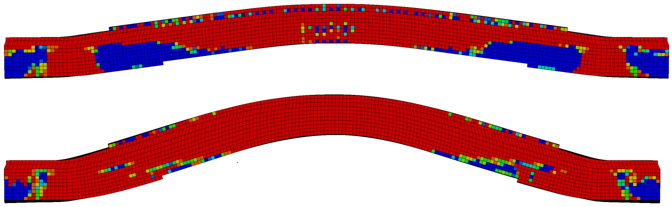


Figure 11: Deformation history for Blind simulation blast test for different fracture energies, 138 and 69 Nm/m^2, and strain rate dependent strength turned on, 1, and off, 0.

6. Discussion

For the study in this paper experiments were searched where structural reinforced concrete members were loaded by blast waves. The response aimed for in this study was deformation due to bending. The number of experiments is scarce due to the high costs related to this kind of loading. In addition, the experiments are aimed at studying the structural resistance of the structure rather than the calibration of numerical models. The consequence of this is that the results extracted are not always adequate for the calibration of numerical models.

For a structural member, such as a beam, the maximum deflection is usually studied from the moment of load application to some time after the maximum deflection of the beam has occurred. The influence of strain rate dependency on the concrete compression strength for CDPM2 has been studied in [2]. The conclusion was that the increase of strength can be too high compared to experiments [10] when 3D continuum elements were used. In the present study, the influence of the strain rate dependency on the deformation of the structure was studied. The deformation is obviously influenced by the strain rate, as seen in Figure 9 and Figure 11. As a reference, no strain rate dependency was chosen neither for the slab nor for the beam simulations. The deformation compared to the blind simulation experiments deviated -10% and decreased by 30% with strain rate dependency. For the beam experiments, the deviation of the maximum deformation was +83% for the reference and +50% with strain rate dependency compared to the experiment.



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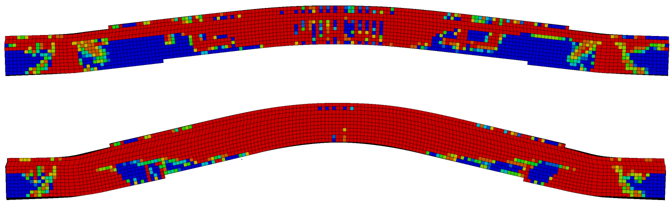

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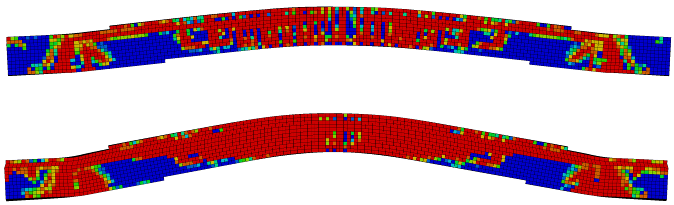

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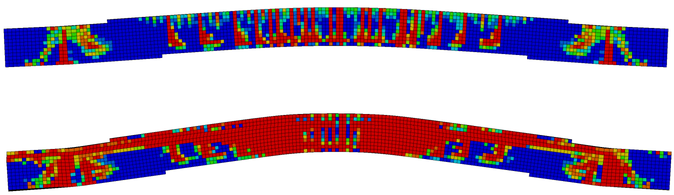

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Figure 12: Contour plot of damage of the Blind simulation blast test slab for different fracture energies, 138 and 69 Nm/m^2, and strain rate dependent strength turned on, 1, and off, 0. Top contour plots at the time of half maximum deformation and bottom plots at maximum deformation. Red colour represent full damage and blue represent no damage.

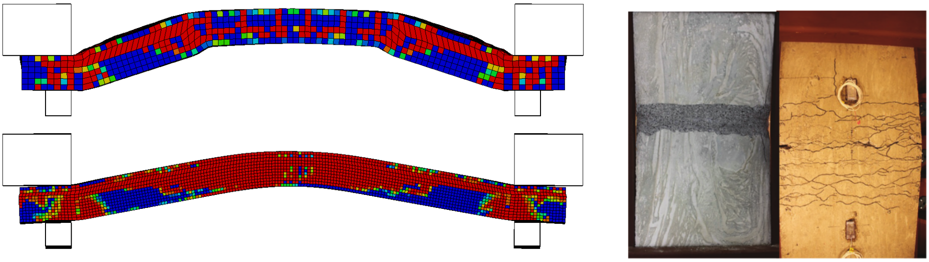

Figure 13: Failure mode before and after mesh refinement and crack pattern seen from the top (left) and the back side (right) of the slab; taken from [5].

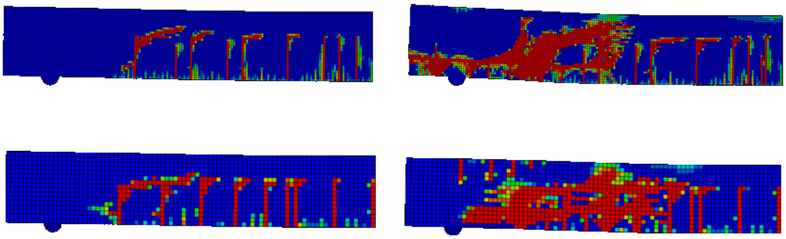

Figure 14: Shear crack near support for refined mesh (above) and for coarse mesh (below) for the FOI beam early (left) and late (right) in the analyses.

The reason for the very large deviation in deformation for the beam is believed to have mainly originated from the severe shear cracking near the support that takes place in the numerical analyses. From the results of the experiment, shear cracks near the support are observed but shear failure as achieved in the analyses does not appear. Since the beam was reinforced with stirrups, the shear cracks that appear will be kept together and create two planes that slide with a normal force created by the stirrups. Thus, a sliding shear crack, or a mode 2 crack, will probably not be represented very well by this type of numerical model.

The influence of fracture energy on the deformation and failure mode has been studied before in [4]. The conclusion was that the fracture energy influences the failure mode. In particular, the reduction of the fracture energy was a way to achieve shear cracking by the support. In the present study, the influence of the fracture energy was studied with regard to the deformation. In order to have a firm base, a reference value was computed according to [21]. This resulted in a fracture energy of 138 Nm/m^2 for the blind simulation slab. Compared to the slab experiments, the deformation deviated -10%; for a fracture energy half of the initial value, a +37% deviation was observed.

For the beam simulation, the development of a shear crack is observed. This shear crack is also observed in the experiment. However, the influence of this shear crack is much greater in the simulation than in the experiments. The conclusion is that this kind of structural response is not possible to simulate more realistic using the current constitutive model. From the results for the blind simulation test slab, the effect of including strain rate dependent material strength had less influence on the deformation of the slab compared to the difference in deformation when half the initial fracture energy was used. A similar observation can be seen for the FOI beam but because of the large difference in deformation due to the shear crack near the support, the inability to represent that type of failure becomes more pronounced.

7. Conclusions

The aim of this study has been to evaluate the performance of CDPM2 when used to model reinforced concrete structures subjected to blast loads. Previous studies that have been performed have shown that the strain rate dependent strength might overestimate the capacity. Furthermore, it has been shown that the ductility of the concrete during tensile fracture might be overestimated due to the constitutive laws that describe the development of fracture energy for higher strain rates. To study these features, different initial values of the fracture energy were used in the analyses. The effect of the strain rate dependency was studied by including the effects of high strain rates and by turning off this feature for CDPM2.

It was found that the strain rate dependency had a moderate effect on the deformations and the damage of the concrete structures for the flexural response that was evaluated. However, it was found that the initial value that was chosen for the fracture energy had a much larger influence on the flexural deformations. Thus, it is necessary to develop a better understanding for how fracture energy should be chosen when numerical models of dynamic events are studied.

The study of the FOI beam shows that flexural shear failure cannot be adequately described given the type of numerical model used in this study. It was also shown in the model for the slab in the Blind simulation blast test that some modification had to be made when small elements were subjected to large compression stresses. In order to treat the very brittle response due to the description of compression softening in the current implementation of CDPM2, elements close to the support and elements in the middle of the loaded side of the slab were replaced by elastic elements. Furthermore, the element mesh had to be refined in order to reduce the effect of delamination in the concrete near the reinforcement.

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