Blast-resistant Building Design

Methods and solutions for the blast-resistant design of buildings subjected to an LPG tank truck explosion

Final Master Thesis Report

Master Building Engineering

Faculty of Civil Engineering and Geosciences

Delft University of Technology

Delft, 26 March 2009

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TNO

Graduation committee

prof.dipl.-ing. J.N.J.A. Vamberský — Delft University of Technology, Faculty of Civil Engineering and Geosciences, Section Structural and Building Engineering

ir. S. Pasterkamp — Delft University of Technology, Faculty of Civil Engineering and Geosciences, Section Structural and Building Engineering

dr.ir. J. Weerheijm — Delft University of Technology, Faculty of Civil Engineering and Geosciences, Section of Structural Mechanics. TNO Defence, Security and Safety

ir. J.F.M. Wessels MBA — TNO Build Environment and Geosciences

dr.ir. A.H.J.M. Vervuurt — TNO Build Environment and Geosciences

Preface

This is the final Master thesis report of my graduation work at the faculty of Civil Engineering and Geosciences at the Delft University of Technology. The Master thesis forms the final step to obtain the degree of Master of Science in Civil Engineering. The specialization within my Master is Building Engineering.

The study is partially carried out at TNO, the Dutch organization for applied scientific research, at the unit Build Environment and Geosciences, located in Delft.

I would like to thank my graduation committee for their help and guidance.

Max Janssen

Delft, March 2009

Summary

Because of the wish for, and the necessity of the multiple use of space the amount of tunnels and buildings in close proximity of roads are increasing. Meanwhile the number of transports of dangerous goods is also increasing. Therefore it would be desirable to have a highway network which is fully accessible for the transport of dangerous goods. In order to make this possible the effect of an incident with dangerous goods on the buildings along the road should be known. This study focuses on the effect of an explosion blast on a building. It aims to propose a method to determine this effect, and to propose several structural solutions for buildings, situated alongside the road, to make them more resistant against the blast load from an explosion on the road as a result of an incident with an LPG tank truck.

Of all the possible situations on the Dutch highway network some are discussed and three are chosen to study the effect of an explosion. These situations are: in the tunnel, at the tunnel mouth and on the open road with buildings beside the road.

Of all types of explosions and effects that can occur, this study only focuses on the effect of a blast wave on a building as a result of a BLEVE of an LPG tank truck of 50 m^3. A BLEVE is a Boiling Liquid Expanding Vapour Explosion. The to liquid compressed gas will evaporate explosively when the vessel ruptures, which results in a blast wave. The overpressure at the building can be schematised by in instant increase of the pressure to the peak overpressure, after which it decreases linearly to zero after a certain positive phase duration. The impulse is the surface under the pressure-time curve.

An explosion in a tunnel results in the building-up of pressure. To determine how the overpressure exits the tunnel and reaches the building, an estimation of the increase of volume and the wave front surface is made. For an explosion in the open field five different methods are discussed. These range from a TNT equivalence method to a gas-dynamic modelling. The calculated values for the overpressure differ a lot and not all methods give a value for the impulse. Based on the described methods it is concluded that an explosion on the open road gives a larger blast load on the building than an explosion in the tunnel or the tunnel mouth.

The blast-wave of an explosion of an LPG tank truck can cause damage to the buildings. Several aspects of the blast-wave are discussed, and how they determine the load that is transferred to the main load bearing structure. These aspects are among others the reflection, dynamic blast load, adjacent buildings, distribution of the blast load on the façade of a building, and glass failure. For several of these aspects methods are given to determine the load on the building. The peak reflected overpressure on a defined model building of 35 m high is 101 kPa and the impulse is 1,97 kPa*s.

After this the mechanical properties of a building as a whole and of the building elements that determine its response to this load are determined. These are the static strength, natural frequency and ductility. After the loading, all the response aspects and the structural properties are determined and an estimation of the damage to the building and its elements is given with methods that calculate the chance of collapse, the equivalent static load and the required ductility.

From sensitivity analyses of the damage methods it is concluded that it is possible to make a building able to withstand the blast from an LPG explosion by choosing the right parameters. The most sensitive parameters of the blast load and the mechanical properties of the building are the tank volume, the distance to the explosion, the horizontal design load and the height of the building. A higher building results in a larger design load and a smaller angular rotation.

Sensitivity analyses of the methods that determine the damage to the structural elements of the model building have shown that the glass is the first to fail but the first structural elements to fail are the columns and walls perpendicular to the blast load direction. The distance from the explosion within which an element of a building (and thus the whole building) will fail is called the collapse radius. The collapse radii of the elements of the model building are:

	Collapse radius
Global building	58 m
Glass	363 m
Core	24 m
Floor	<1 m
Column	100 m
Perpendicular wall	83 m
Non perpendicular wall	9 m

Table 0-1: Collapse radii model building elements

By changing the parameters of the elements the collapse radius can be reduced. Because the column and the perpendicular wall are the governing elements the parameters that should be changed are the column or wall width, the amount of reinforcement in the concrete and the concrete quality, decreasing the modulus of elasticity.

A building that will not collapse at a distance of only a few meters from the explosion is possible. In order to achieve this it is necessary that no walls perpendicular to the blast load, no separate columns and no beams are used. Furthermore, instead of using the wind load as the horizontal design load (in area I according to NEN6702) a larger horizontal design load should be used, which is the wind load multiplied by an explosion factor which is introduced here. This explosion factor depends on the building height (just as the wind load) and the distance from the explosion (the collapse radius) which can be read from figure 0-1. The explosion factor is a tool for blast-resistant building design.

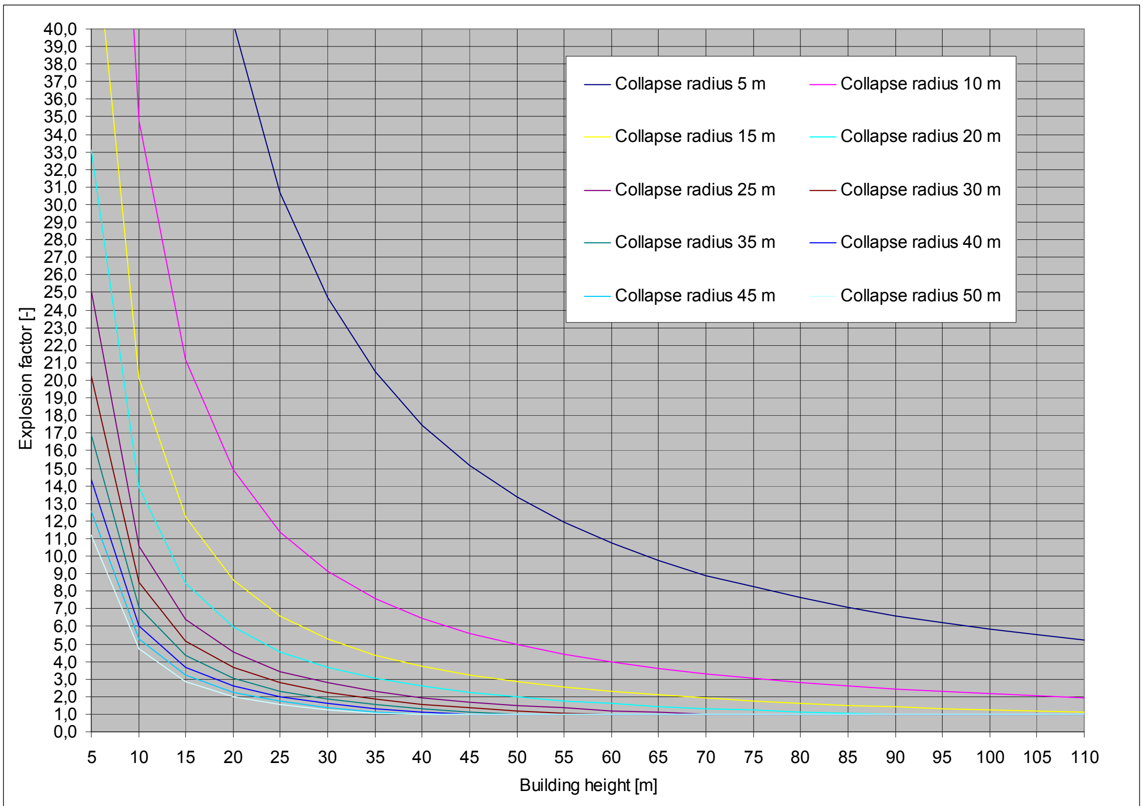


Figure 0-1: Graph explosion factor

Glossary of terms

Dangerous goods
Substances or mixtures of substances, which because of their intrinsic properties or the circumstances in which they occur, form a danger for men or environment, as a result of which damage to health or life can be inflicted.

ADR
The 'Accord Européen relative au transport international des marchandises dangereuses par route'. A European agreement on the transport of dangerous goods on the road, which is based on guidelines by the United Nations. The ADR distinguishes thirteen classes of dangerous goods.

BLEVE
BLEVE is an acronym for Boiling Liquid Expanding Vapour Explosion. It is an explosion and it occurs when a vessel with liquefied gas ruptures and the liquid instantly evaporates, causing a rapid increase of volume.

Blast
Because of the rapid increase of volume an explosion is always accompanied with an increase of air pressure. This air pressure is higher than the atmospheric air pressure and therefore performs a load on anything in its range. The surrounding air is pushed away and the air pressure propagates, this produces a blast-wave.

Peak side on overpressure
Maximum overpressure of the blast that reaches the building.

Positive and negative phase duration
The positive and the negative phase duration are respectively the time that the blast performs a positive and a negative pressure.

Peak reflected overpressure
Maximum reflected overpressure of the blast on the building, which is the actual load on the building

Impulse
The impulse is the area under the pressure time curve of the blast wave.

Static strength
The static strength of a building is the ultimate strength which is the ability of the building to resist a horizontal working load on the façade of the building without collapse of the building, but with permanent deformations.

Natural frequency
The structure will begin to vibrate as a result of the load. The value of the natural frequency says something about the speed with which the structure responds to the load.

Angular rotation
The angular rotation is the radial frequency and is a measure of the rotation rate. It is the frequency of the rotation of the building.

Ductility
The ductility is a measure for the ability of a structure for plastic deformation. It is the quotient of the maximum deformation and the maximum elastic deformation.

Collapse radius
Distance from the explosion source within which failure of a structural element will occur.

Explosion factor

Given the criteria that the building structure does not have walls perpendicular to the direction of the blast and no separate columns and no beams the building can be considered not to collapse when the horizontal wind load (in area I from NEN6702) is multiplied with this factor.

Table of content

1 Introduction

1.1 Problem description

The density of the population and the industry in the Netherlands is very high compared with other nations. The amount of space is therefore scarce and thus increasingly used intensively. This urbanisation asks for a multiple and efficient use of space. Buildings are constructed just beside the road, leaving little space between the road and the buildings. Also buildings are constructed over the road, which creates a sort of tunnel.

Because of the contacts with the industries of the neighbouring countries the transport network is intensively used. The transport of dangerous goods has increased and will increase further in the future. A number of restrictions apply on the Dutch national highways for the transport of dangerous goods, such as LPG. Several tunnels are not open to this type of transport and therefore alternative routes are used. From an economic point of view, a national highway network which is fully open to the transport of dangerous goods would be preferable.

Because of the wish for, and the necessity of the multiple use of space the amount of tunnels and buildings in close proximity of roads are increasing. Meanwhile the number of transports of dangerous goods is also increasing. Therefore it would be desirable to have a highway network which is fully open to the transport of dangerous goods. In order to make this possible the effect of an incident with dangerous goods on the buildings along the road should be known. This study focuses on the effect of a blast on a building in the vicinity of a tunnel as a result of an explosion on the road, in the tunnel or just outside the tunnel.

1.2 Objectives

Objective:
This study aims to propose a method and solutions for buildings, situated alongside the road, to make them more resistant against the blast load from an explosion on the road as a result of an incident with a transport of dangerous goods.

Several sub questions are distinguished:

- In what situation will the explosion take place?
- How can the explosion load be determined and how reliable are these methods?
- What is the difference in the explosion load on a building resulting from an explosion in a tunnel or on the open road?
- What is the effect of the design of the building on the load on the building (on its main load bearing structure)?
- How much load can a building take and what will the amount of damage be?

These questions will be answered in this report.

1.3 Approach

A global approach of the study and the structure of the report are presented here.

In chapter 1 a general introduction is given of the problem and the objectives are defined. And an approach of the study is given.

Chapter 2 gives a description of the situation in which the explosion takes place. What is the configuration of the road and the buildings along the road?

The type of explosion that will take place and how to determine the load form such an explosion on the buildings is described in chapter 3. With this knowledge the load from the explosion in the situations defined can be determined. The chapter is concluded with the definition of the design load.

Chapter 4 presents a method to determine the effect of an explosion on a building. First several aspects that affect the load of an explosion on a building are described, such as reflection against the building and the glass failure. After that, methods to calculate the strength of and damage to the building and its elements are given.

In chapter 5 the calculated load from the explosion from chapter 3 and the response of the building from chapter 4 is combined to determine the response of a defined model building.

As a result of the outcomes of chapter 4 and 5 conclusions are drawn and recommendations are given in chapter 6 on how to improve the design of the building to make it resistant to the design blast load.

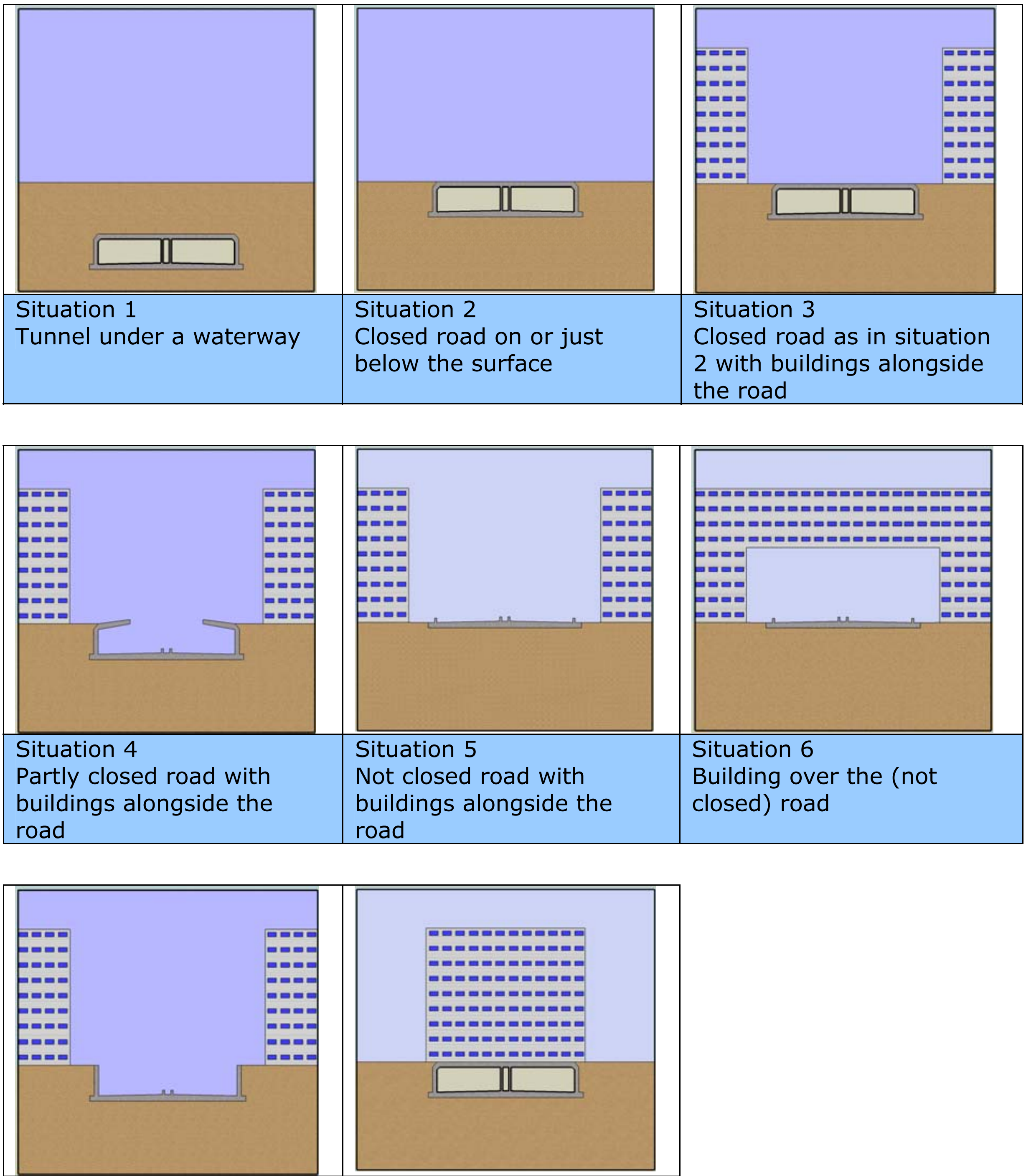
2 Situation description

In order to propose blast-resistant design solutions for buildings, the load on a building from an explosion on the road has to be known. Before this explosion load is calculated in chapter 3, a description of the situation where the explosion takes place is defined in this chapter.

A lot of different situations can be distinguished on the Dutch highway. The orientation of the road, the tunnel and the building determine the eventual explosion load on the building.

Not all situations will be covered in this study. In order to answer one of the sub questions defined in paragraph 1.2 ‘Objectives’ about the difference of the explosion load on a building from an explosion in a tunnel or on the open road, the situation that will be discussed here has to have a part open road and a part tunnel with buildings beside the road. For this study the situations of interest are those where buildings are close to the open road and close to a tunnel. Examples of such situations in the Netherlands are the closed road in the A2 near Leidsche Rijn, the Utrechtse Baan in The Hague, the plans for the Zuid-as in Amsterdam, all tunnels where the transport of explosion dangerous goods is restricted - such as the Drechttunnel - and the plans for a closed road in the A28 near Leusden.

Eight situations have been set up that represent the situations on the Dutch highway.



Situation 1
Tunnel under a waterway

Situation 2
Closed road on or just below the surface

Situation 3
Closed road as in situation 2 with buildings alongside the road

Situation 4
Partly closed road with buildings alongside the road

Situation 5
Not closed road with buildings alongside the road

Situation 6
Building over the (not closed) road

Situation 7
Sideway closed road with buildings alongside the road

Situation 8
Building on the tunnel

Figure 2-1: Situations

On basis of what will happen in these eight situations when an explosion takes place, a selection is made to determine which of the situations is the most interesting to work out in the scope of this study. To select a situation the situations are further looked at below.

Situations 1 and 2 are identical, except for the layer of soil or water on top of the tunnel in situation 1. The load of the soil attributes to the resistance of the tunnel to an explosion. Because the effect of the soil on the explosion is not taken into account in this study situation 1 is disregarded.

When the road has a canopy as in situation 4 there exists the danger of the canopy failing and banging against the building. Besides this disadvantage the canopy can also act as a protective shield for the building. Such an obstacle in the path of an explosion will cause a 'shadow' behind the canopy where a lower pressure will occur. This shielding effect will not be discussed in this study.

Also situation 6 and 8 show some resemblance. In the situation of a building on the tunnel the concrete structure of the tunnel will act as a protective barrier for the building on top. When the blast is directly under the building, as in situation 6, no such barrier exists, so a larger blast is to be expected. These situations with buildings over the road or tunnel are not within the focus of this study. Only the effects of explosions on buildings beside the road are discussed.

Situation 5 and 7 are very similar. The difference between the two is that in a deepened road the debris caused by the explosion will impact in the concrete structure beside the road and with the situation of a road on surface level the debris will impact in the nearby building. The effect of debris will only be discussed briefly in chapter 4.2. Also the pressure load that reaches the building will be lower with a sideways closed structure. This effect of a deepened road on the explosion load on the building will be discussed in chapter 4.2.

Because of the confined space in a tunnel the pressure as a result of an explosion can reach very high levels. The overpressure from an explosion in a tunnel travels through the tunnel, exits the tunnel and reaches the buildings just outside the tunnel mouth. The hypothesis is that because of this pressure built-up an explosion in the tunnel can generate even a higher load on the building (next to road outside the tunnel) than an explosion on the open road in front of the building, where there is a large open space where the pressure can fade out. To find an answer to this hypothesis the load on a building is calculated in the following three situations:

- Explosion in the tunnel
- Explosion in the tunnel mouth
- Explosion on the open road

These are the three scenarios for which the load on the building is determined in chapter 3. When these loads are calculated the governing situation for the explosion load on a building is known.

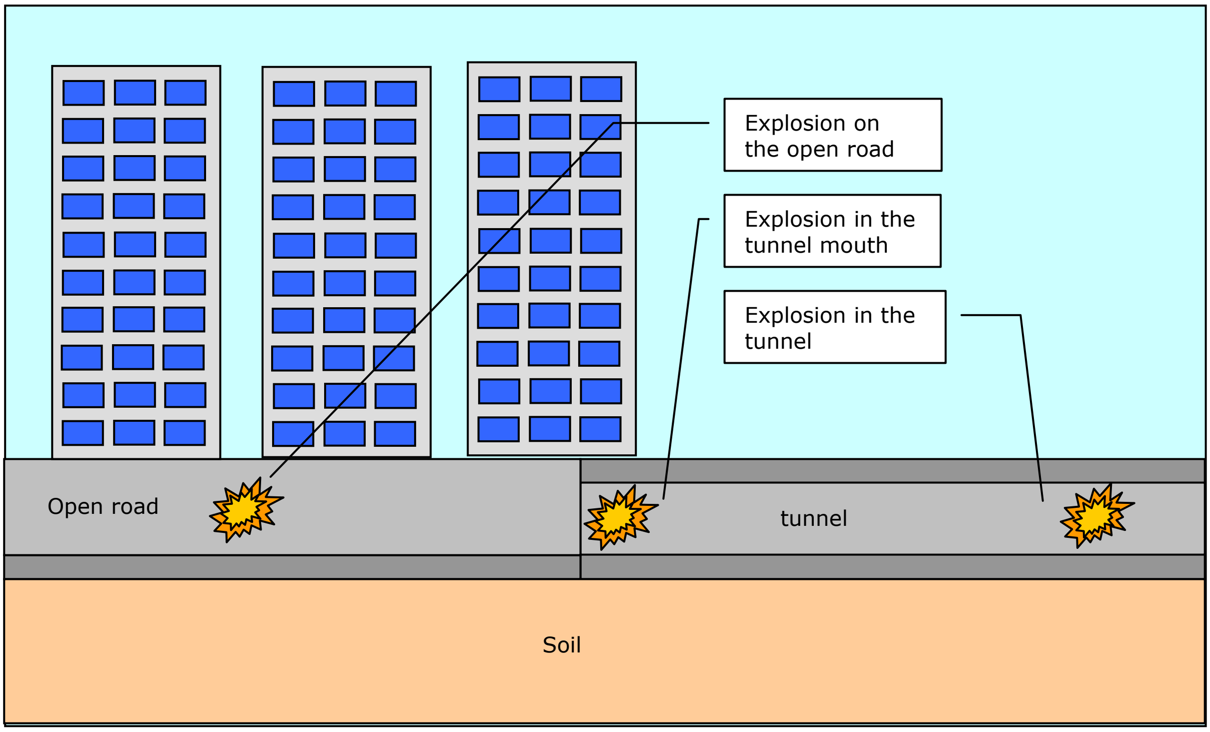


Figure 2-2: Cross section three load scenarios

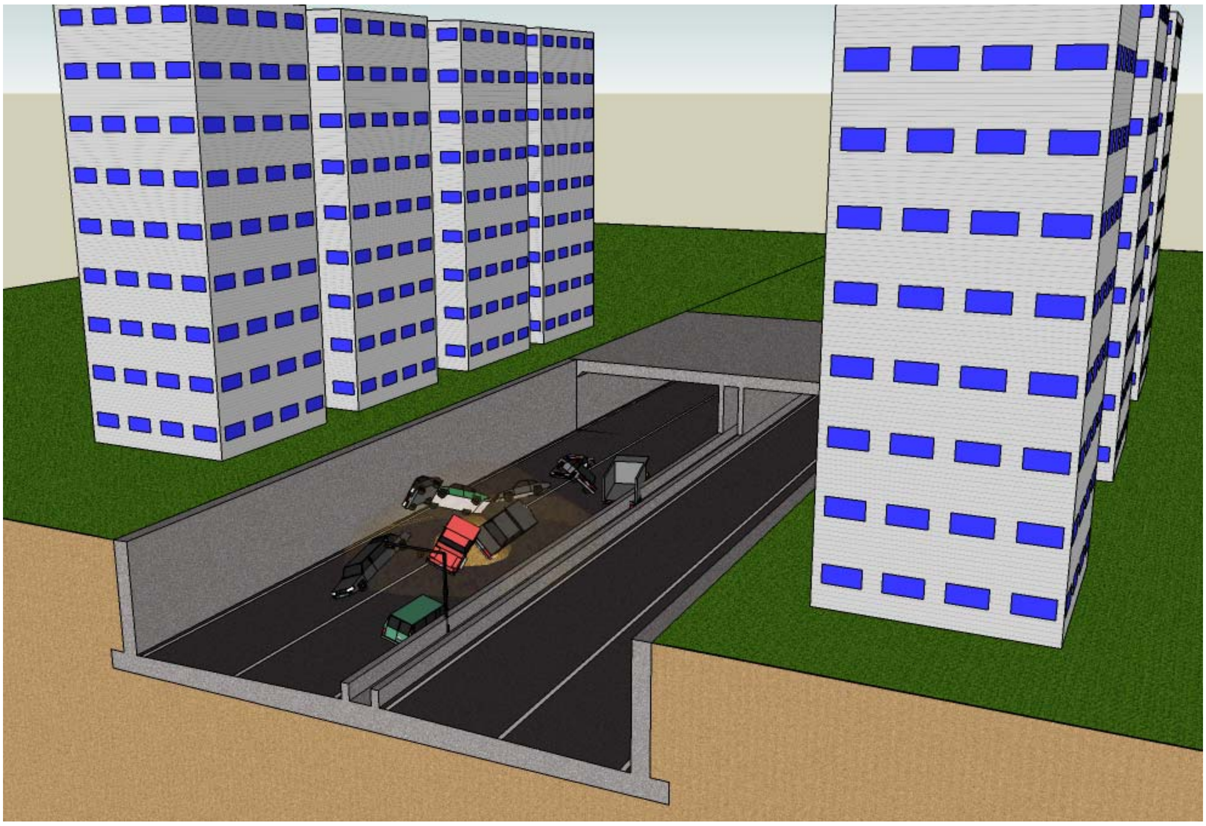


Figure 2-3: 3D view of explosion on the open road

As stated in the objectives the focus of this study is on the response of the buildings. The structural response of the tunnel is not considered. Whether the tunnel will collapse, and what the effect of such a collapse will be on the course of the pressure wave in the tunnel and the load on the building is a separate study and is not within the scope of this study. For an explosion in a tunnel it is assumed that the tunnel structure will not fail.

In order to calculate the load on the building the distance from the centre of the explosion to the façade of the building has to be known. For this research a distance of 20 m is assumed. For the cross section of one tube of the tunnel 72 m^2 is assumed.

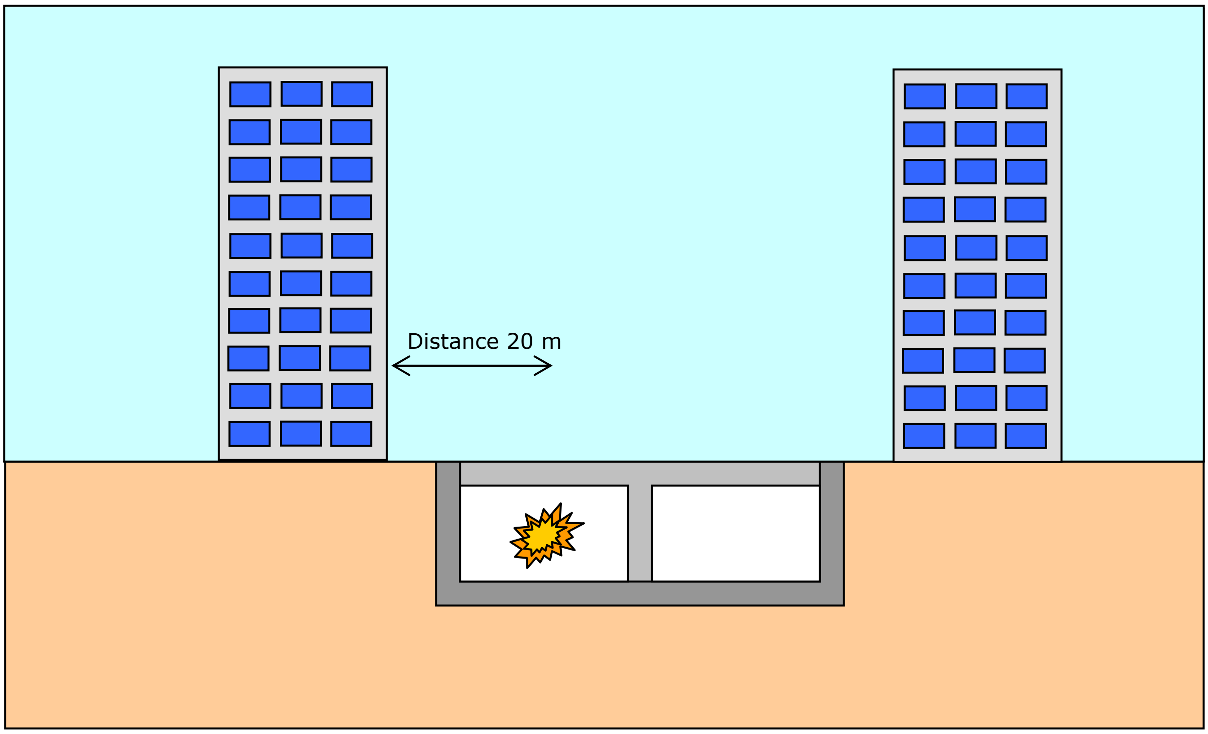


Figure 2-4: Distance building form explosion

For the building there are some variables that remain. These are for example the geometry, such as the height of the building and the material of the structures. Is it a steel or concrete structure? Another one is the distance between the road and the building. These aspects are discussed in chapter 4.2.

3 Explosion load

To determine the response of a building to an explosion on the road, the load of the explosion has to be known. This chapter gives an answer to what kind of explosion takes place, and how the explosion load can be determined for the three different scenarios; an explosion in a tunnel, an explosion in the tunnel mouth and an explosion on the open road.

3.1 Introduction

The definition of an explosion and all types of explosions are given. For this study only one specific explosion is considered and chosen in this paragraph.

3.1.1 Definition

According to three sources the definition of an explosion is the following:

- Wikipedia [25]:
  An explosion is a sudden increase of volume and release of energy in an extreme matter, usually with the generation of high temperatures and the release of gasses. An explosion creates a shock wave.

- Oxford English dictionary [1]:
  Explosion
  1 An act of exploding
  2 A sudden increase in amount or extent
  Explode
  Burst or shatter violently as a result of the release of internal energy.

- Cambridge English dictionary [2]:
  Explosion
  When something such as a bomb explodes
  Explode (burst)
  To (cause to) burst violently

Combining these definitions the characteristics of an explosion are a violent burst, very rapid increase of volume and release of internal energy in an extreme matter and is usually accompanied with high temperatures, flames, the release of gasses, clouds of smoke, loud noises and a blast wave.

3.1.2 Dangerous goods

In the Netherlands a number of products and substances are produced and handled (sometimes in large quantities) which could be marked as dangerous goods. Examples of producers and industries working with dangerous goods are the chemical industry, fuel industry and the army.

The following definition for dangerous goods is used by the Dutch advisory council for dangerous goods:

‘Substances or mixtures of substances, which because of their intrinsic properties or the circumstances in which they occur, form a danger for men or environment, as a result of which damage to health or life can be inflicted.’

Dangerous goods can be categorized in different classes according to the chemical characteristic producing the risk. On 30 September 1957 the 'Accord Européen relative au transport international des marchandises dangereuses par route' [3] became active. This is a European agreement; know as ADR, on the transport of dangerous goods on the road, which is based on guidelines by the United Nations. The ADR distinguishes thirteen classes of dangerous goods, these are presented here. It is a widely applied categorization but some countries' regulations have some variations. For example 'Annex I of the Council Directive 96/82/EC' of 9 December 1996 [4] introduces a different categorization. In this Council Directive more attention is paid to the danger and to mixtures of dangerous goods than in the categorization that is given here.

Class 1.	Explosives Explosive substances and objects. Examples: dynamite, TNT, nitro glycerine, fireworks, ammunition, rocket propellant
Class 2.	Gasses Compressed, liquidized or pressurized gasses Examples: Acetylene, hydrogen, cryogenic gasses/liquids, fluorine, chlorine, LPG
Class 3.	Flammable liquids Examples: petrol, diesel, kerosene.
Class 4.1	Flammable solids Example: magnesium
Class 4.2	Spontaneously combustible solids Solids susceptible to self ignition Example: white phosphorus
Class 4.3	Dangerous when wet Substances which, when in contact with water, develop combustible gasses Examples: sodium, calcium
Class 5.1	Oxidizing agent Substances which stimulate combustion Example: Ammonium nitrate
Class 5.2	Organic peroxides Example: benzyl peroxides
Class 6.1	Toxic substances Examples: cyanide, pesticide
Class 6.2	Biohazard Infectious substances and samples Examples: virus cultures, used intravenous needles
Class 7	Radioactive Examples: uranium, plutonium
Class 8	Corrosive substances Examples: sulphuric acids, sodium hydroxide

Class 9 Miscellaneous substances
Other substances which can be dangerous for man or environment
Examples: asbestos, dry ice

3.1.3 Explosion dangerous goods

As stated, this study aims to give blast-resistant design solutions for buildings under the influence of an explosion as a result of an incident with the transport of dangerous goods. Of all the classes of substances listed not all can cause an explosion and are therefore relevant for this study.

The substance that potentially causes the most destructive explosion determines with what loads a structure has to be dimensioned in order enable it to withstand an explosion. This depends on the volume in which it is transported and the explosion strength of the substance.

The most destructive substances are the explosives in class 1, which are almost exclusively used by the army. However the general restrictions on the transport of dangerous goods do not apply to the army. Also fireworks are categorized in class one, for which a separate legislation applies. Therefore in this study the class 1 explosives are not taken into account.

The other classes that can cause an explosion are class 2. gasses, class 3. flammable liquids, class 4.1 flammable solids, class 4.2 spontaneously combustible solids and class 4.3 dangerous when wet. Of all the substances, the potentially most destructive transports are flammable gasses and liquefied or pressurised gas. A common example of such a liquefied gas transport are LPG tank trucks. The tank volume can be up to 60 m^3. LPG is a usually a mixture of propane and butane.

Figure 3-1: LPG tank truck

3.1.4 Type of explosion

In general two types of explosions can be distinguished: physical and chemical explosions. A chemical explosion, such as a gas explosion, is an oxidation reaction that produces large amounts of hot gas. A physical explosion is the bursting of a sealed container as a result of internal pressure increase or vessel failure. A BLEVE is an example of a physical explosion but it can also result in a chemical explosion when the gasses ignite.

Gas explosion

A gas explosion is a rapid and violent oxidation reaction that produces large amounts of hot gas. When a vessel is mechanically damaged and a highly flammable gas starts leaking it can mix with the air and form an explosive mixture. When the substance does not ignite right away the cloud will be larger and form a more optimal mixture, the explosion will be larger. Also the draught influences the scale of the explosion. For a chemical explosion two different types of combustion (an oxidation reaction) can be distinguished: deflagration and detonation. Detonation is a self sustained supersonic combustion generating a shock wave due to energy release in a reaction zone behind it. In a detonation the shock compresses the material increasing the temperature to the point of ignition. The ignited material burns behind the shock and releases energy, which supports the shock propagation. Across the shock there is always a rapid rise of pressure, density and temperature. The reaction rate lies between 1500 and 9000 m/s. A deflagration propagates at a rate much lower than a detonation (up to 800 m/s). The chemical reaction is maintained by the transport of heat. Because the higher pressure generated by a detonation, they are usually much more destructive.

Figure 3-2: Gas explosion after a collision of a car into a gas pump station [27].

BLEVE

BLEVE is an acronym for Boiling Liquid Expanding Vapour Explosion. It is an explosion and it occurs when a vessel with liquefied gas ruptures and the liquid instantly evaporates, causing a rapid increase of volume. A BLEVE can occur when a tank containing a liquefied gas ruptures as a result of a collision. In this case it is called a cold BLEVE. A BLEVE can also occur when a vessel endures an internal pressure build-up as a result of heating of an external fire (following the basic gas laws), in that case it is called a warm BLEVE. A tank is always equipped with pressure release valves to release the increased pressure. When the capacity of the valves is exceeded the tank can rupture releasing the liquid to atmospheric pressure where it will instantly vaporize causing a very rapid increase of volume. Additionally the gas can ignite causing extra damage. The ratio between the volume of liquid and gas is typically around 1:250 for LPG, a common liquefied gas. Liquefied Petroleum Gas normally consists of propane and/or butane. For pure propane liquid gas the volume ratio is 1:300. Under atmospheric pressure propane has a boiling point of 231 K. In order to keep propane liquid at 300 K the substance has to be stored at 1000 kPa or 10 bar.

Figure 3-3: A tank truck enduring an external fire. The pressure release valves fully functioning. Fire-fighters are trying to extinguish the fire and cool the tank by spraying water over it [27].

Figure 3-4: On the left a shot of fire-fighters at an estimated 100 m from a crashed tank truck with a large fire. On the right, only fractions of a second after the shot on the left, a BLEVE occurs with a fireball with a diameter of over 200 m. The fire-fighters duck away [27].

Figure 3-5: On the left one of two large pieces of a tank landed against a house at 125 m distance from an explosion and on the right the other part at 80 m from its original position. The engine of the truck was found at 257 m distance form the accident. (Catalonia, Spain, on 22 June 2002) [26].

Gas explosion vs. BLEVE

Gas explosions are very complex phenomena. There are a lot of parameters that are hard to quantify that determine the force of the explosion. This is for example the time that the gas has been leaking out of the tank; the longer it leaks the bigger the volume of flammable mixture in the tunnel and the bigger the explosion. Also the geometry of the tunnel and the objects on the road, such as (the amount and size of) other cars, in a large extent determine the progression of the explosion. These parameters are hard to predict or determine. The power of a BLEVE can be determined with more certainty. It is presumed that the chance of a very large gas explosion which is in power comparable with a BLEVE has a much smaller change of occurrence then a BLEVE. Although the chance of a small gas explosion is much larger then a BLEVE, the risk of a BLEVE is larger because of its larger strength. Because of the complexity of gas explosions, the larger risk of BLEVE's, the better availability of literature on BLEVE's than on gas explosions and to narrow the scope of this study only BLEVE's are considered in this report.

3.1.5 Explosion effects

Several effects accompany an explosion. These are high temperatures, fire, air blast, ground shock and crater formation, clouds of smoke, loud noises and debris [5].

High temperatures can cause considerable damage; it can cause spalling of the concrete. Heat however is characteristic for fire and therefore extensively described in existing literature. The heat duration in an explosion is relatively short in relation to a standard fire. The effect of heat load, fire and smoke is not considered in this study. When the explosion takes place close to, on or in the ground, the ground will vibrate similar to an earthquake but with a different frequency. This ground shock will influence the foundation of the buildings. Also a crater can form on the place of the explosion. As the response of the soil is out of the scope of this study, this will not be discussed. The effect of debris also will not be discussed.

Because of the very rapid increase of volume the pressure in the air increases and a blast wave occurs. It is assumed that the blast is the dominating effect of a BLEVE for structural damage. The only aspect that will be discussed in this study is the air blast or air overpressure.

3.1.6 Conclusion

Of all possible types of explosions and effects that can occur, this study will only focus on the effect of a blast wave on a building as a result of an explosion of an LPG tank truck of 50 m^3. For the calculations the properties of propane at 326 K will be used.

3.2 Methods for determining the blast load

First the theory of a blast wave is discussed [6, 7]. After that the explosion load on a building is determined for the three different scenarios in chapter 3.2.2, 3.2.3, and 3.2.4, where several methods and theories will be discussed for each scenario. The used methods are all based on guidelines, literature and recent studies.

3.2.1 Characteristics of a blast wave

Because of the rapid increase of volume an explosion is always accompanied by an increase of air pressure. This air pressure is higher than the atmospheric pressure and therefore performs a load on anything in its range. The surrounding air is pushed away and the air pressure propagates, this produces a blast-wave or pressure-wave. Because the wave propagates faster than the air pressure rises, any pressure wave will adopt the shape of a shock wave as the distance grows.

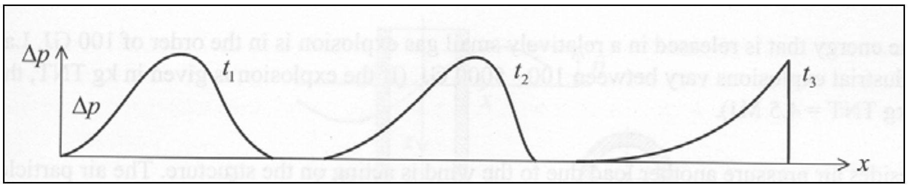


Figure 3-6: Development of a shock wave (t3 > t2 > t1)

A pressure wave is a propagating increase of pressure. Large explosions such as a BLEVE cause a shock wave. A shock wave is instantaneously; the rise time is zero, the maximum overpressure is reached at t=0; the shock front. For other pressure waves the maximum pressure is only reached after a certain rise time. The maximum pressure is the peak overpressure P_s. After the peak overpressure the overpressure will reduce to zero. For shockwaves also a negative pressure phase can be distinguished. This under pressure forces a negative load on the obstacles in its path. The negative phase is usually neglected.

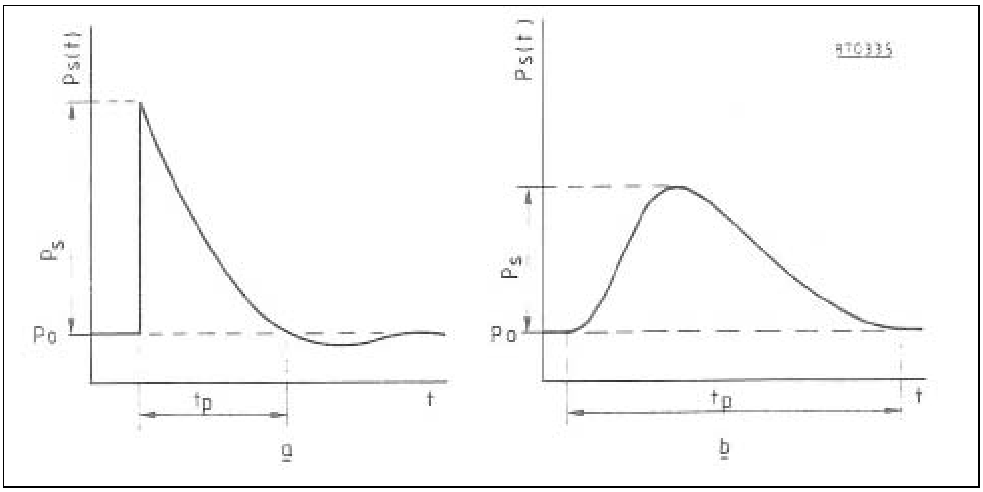


Figure 3-7: Characteristic pressure-time diagram of: a. a shockwave and b. a pressure wave.

Besides the peak overpressure and the phase duration t_p the shockwave is also characterised by the impulse I_s, the area under the pressure-time curve.

The amplitude of the shockwave at a certain distance from the explosion is damped:

$$X(x) = X_0 e^{(-\gamma x)}$$

Where

X(x)	= amplitude at a distance x	[m]
X_0	= amplitude of the wave at a source	[m]
γ	= damping ratio	[-]

In the open field a pressure wave can expand freely in all directions and therefore it will reduce in strength more quickly than an explosion in a tunnel. In a tunnel there is hardly any damping.

The shockwave can be simplified by a course of straight Lines. Characteristic are the shape (shock or pressure wave), atmospheric pressure P_0, peak overpressure P_s, Positive phase duration t_p and the impulse I_s = 0,5* P_s *t_p.

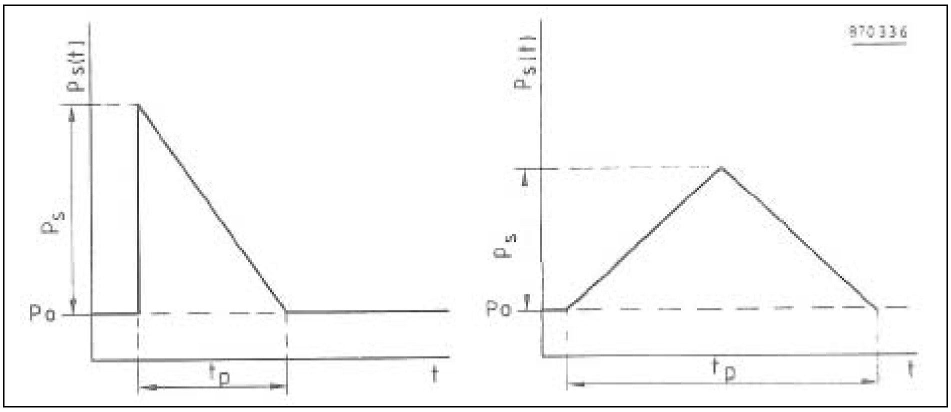


Figure 3-8: Schematized pressure-time Lapse of a shockwave (left) and a pressure wave (right)

3.2.2 Scenario 1: Explosion inside the tunnel

When an LPG tank truck explodes inside a tunnel the pressure builds up. A tunnel forms a one-dimensional situation, forcing the blast in two opposite directions. The pressure wave will exit the tunnel, where there are buildings 20 m from the road. The overpressure that reaches the building outside the tunnel will be lower than the initial overpressure and will act acts as a load on the building structure. The initial overpressure on the location of the explosion and the overpressure that reaches the building are determined here. Two methods to describe the initial overpressure in the tunnel are discussed. The overpressures will be expressed in kPa.

Shock-tube-formula method (Liepmann and Roshko, 1967) [8]

This method describes the initial overpressure after the failure of a membrane between a low and a high pressure section. It is based on a gas and not on overheated liquid.

$$\frac{p_1}{p_a} = \left(\overline{P}_{so}+1\right)\left[1-\frac{\left(\gamma_1-1\right)\left(a_a/a_1\right)\overline{P}_{so}}{\left[2\gamma_a\left\{2\gamma_a+\left(\gamma_a+1\right)\right\}\overline{P}_{so}\right]^{1/2}}\right]^{-2\gamma_1/(\gamma_1-1)}$$

Where:

p_1	= initial pressure of gas in tank	[kPa]
p_a	= surrounding (atmospheric) pressure	[kPa]
$\overline{P}_{so}$	= dimensionless initial blast overpressure	$=\left(p_{so}/p_a\right)-1$
P_{so}	= initial peak pressure blast	[kPa]
γ_a	= ratio specific heat of air	[-]
γ_1	= ratio specific heat of liquefied gas	[-]
a_a	= speed of sound in surrounding air	[m/s]
a_1	= speed of sound in liquefied gas	[m/s]

The results of this formula for propane, butane and ethane are given below.

Tabel 4a: Verzadigingsdruk ethaan afhankelijk van de vloeistof temperatuur alsmede de initiële overdruk blastgolf afhankelijk van bezwijkdruk vat.

Temperatuur vloeibare ethaan (K)	Dampdruk ethaan Bezwijkdruk vat (kPa)	Initiële overdruk blast (kPa)
184	100	0
240	1000	280
267	2000	450
285	3000	600
297	4000	700

Tabel 4b: Verzadigingsdruk propaan afhankelijk van de vloeistof temperatuur alsmede de initiële overdruk blastgolf afhankelijk van bezwijkdruk vat.

Temperatuur vloeibare propaan (K)	Dampdruk propaan bezwijk-druk vat (kPa)	Initiële overdruk blast (kPa)
231	100	0
300	1000	280
330	2000	450
350	3000	600
370	4000	700

Tabel 4c: Verzadigingsdruk butaan afhankelijk van de vloeistof temperatuur alsmede de initiële overdruk blastgolf afhankelijk van bezwijkdruk vat.

Temperatuur vloeibare butaan (K)	Dampdruk butaan bezwijk-druk vat (kPa)	Initiële overdruk blast (kPa)
273	100	0
352	1000	280
389	2000	450

Table 3-1: Initial overpressure according to shock-tube-formula-method

It can be read from these tables that the maximum initial overpressure for propane at 330 K is 450 kPa.

Method Van de Berg

In the TNO Delft Cluster report "Pressure load on a cylindrical tunnel caused by an LPG BLEVE: Influence of the vessel failure time" [9] the course of the pressure in time in a tunnel is given. This recent report (April 2008) contains the latest insights and gives a very realistic quantification of the scenario of an explosion in a tunnel. The pressure and impulse at different distances from the explosion in the tunnel are given. At x=0 the initial peak overpressure is 1280 kPa. This is the maximum pressure and only occurs on the location of the explosion. The results of this study for an LPG tank of 50 m^3 at 326 K are in de table below. An LPG initial temperature of 326 K corresponds to a uniform internal vessel pressure of 1800 kPa. The pressure wave reflects in the tunnel and will fade out eventually in an infinite tunnel. But for this study it is assumed that at the tunnel mouth an overpressure remains of 150 kPa.

Distance Downstream (m)	LPG temperature 326 K		
	Peak overpressure P_s (kPa)	Impulse I_s (kPa*s)	Positive phase duration t_p (ms)
0	1280	52	41
20	109	6.3	58
40	151	13.6	90
60	156	17.6	113
80	152	17.9	118

Table 3-2: Pressure load on a tunnel structure for 50 m^3 LPG BLEVE's for a tunnel cross sectional area of 72 m^2 [9].

This table shows that the positive phase gets longer as the pressure front moves on. The pressure wave damps out; P_s gets lower and t_p larger. At the location of the explosion the positive phase is 41 ms. In the absence of other data a positive phase is assumed near the building of 200 ms. How to determine the overpressure at the building is discussed in the next paragraph.

3.2.3 Scenario 2: Explosion at tunnel mouth

If an explosion of an LPG tank truck occurs in the tunnel right before the exit then the pressure can build just as when an explosion occurs deeper in the tunnel. This scenario results in the maximum overpressure in the tunnel. When the explosion occurs on the edge or just outside the tunnel the initial pressure will be lower. An explosion in the tunnel can generate an overpressure of 1280 kPa. Because an explosion at the tunnel mouth will dissipate energy through the opening it will not reach this value. For the overpressure of the explosion at the entrance of the tunnel a pressure of 700 kPa instead of 1280 kPa is presumed, this is a rough estimate.

The effect that occurs when the pressure wave exits the tunnel mouth is called rarefaction. This is a reduction of the density of the medium. Rarefaction is the opposite of compression. It refers to an area of relative low pressure resulting from a pressure wave. The explosion generates a certain pressure at the location of the explosion. Because of rarefaction a lower pressure than the initial overpressure will reach the façade of the building.

In this study a distance R of 20 m is presumed between the road and the building. This is relatively close to the road. The buildings that will experience the largest force are those right beside the road at the exit of the tunnel.

A method to determine the overpressure outside the tunnel when an explosion in the tunnel mouth occurs is presented here. A part of the pressure wave will exit the tunnel and a part will travel through the tunnel. The pressure wave that exits the tunnel can be considered as a cone-like shape presented below. The angles of the cone are chosen at 45 degrees.

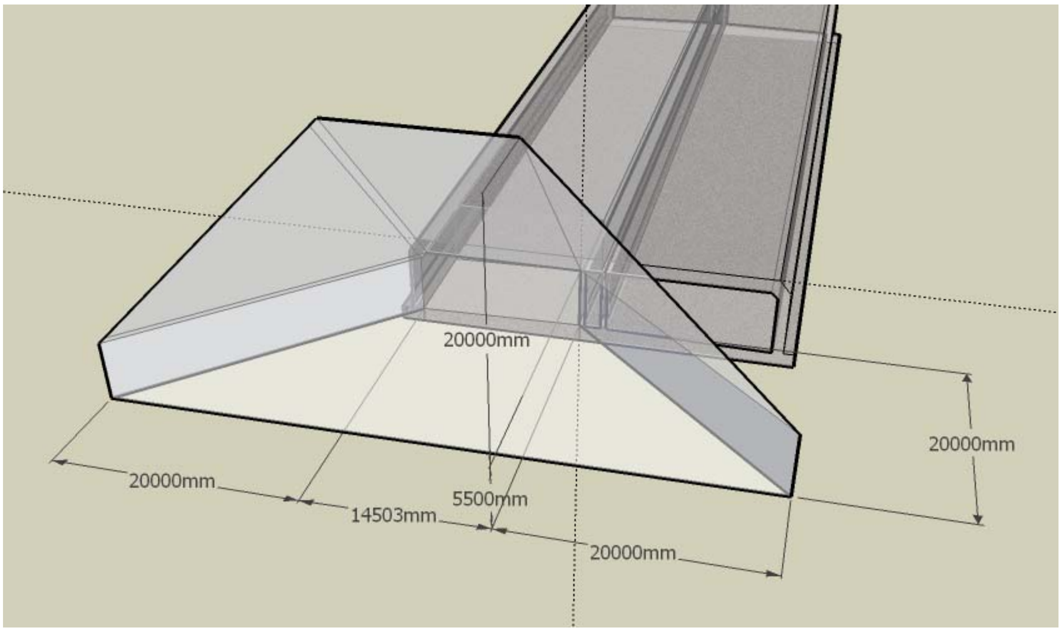


Figure 3-9: Rarefaction at tunnel mouth

The initial overpressure will cover a certain volume in the tunnel. As the wave front has reached the building outside this volume has increased and the pressure has dropped. In this method it is assumed that the pressure in the tunnel is equally distributed over the volume outside the tunnel at the moment that the shockwave reaches the building. An estimation of the overpressure that reaches the building can be made with help of the ideal gas law:

$$p \cdot V = n \cdot R \cdot T$$

Where:

p	= Absolute pressure of the gas	[Pa]
V	= Volume	[m^3]
n	= Number of moles of gas	[mol]
R	= Universal gas constant	[Pa m^3 mol^{-1} K^{-1}]
T	= Temperature	[K]

If n, R and T are constant then the pressure and volume at two moments in time are:

$$p_1 \cdot V_1 = p_2 \cdot V_2$$

If the initial overpressure in the tunnel and the volume in and outside the tunnel are known then an estimation can be made of the overpressure that reaches the building. For the volume V_1 in the tunnel a distance of 10 m and a cross section of 72 m^2 is chosen. Another part of the initial volume will travel through the tunnel. The volume V_2 of the overpressure outside the tunnel is the volume of the cone in the figure. An estimation of the overpressure at the building can now be calculated.

$$V_1 = 10 \cdot 72 = 720 \text{ m}^3$$

$$V_2 = 5 \cdot 20^2 \cdot \tfrac{1}{2} \cdot 2 + 5 \cdot 20 \cdot 14{,}5 + 14{,}5 \cdot 20^2 \cdot \tfrac{1}{2} + \tfrac{1}{3} \cdot 20^2 \cdot 20 \cdot 2 = 11683 \text{ m}^3$$

$$p_2 = \frac{p_1 \cdot V_1}{V_2} = \frac{700 \cdot 720}{11683} = 43 \text{ kPa}$$

In the same way the pressure near the building can be found when the explosion takes place deeper in the tunnel. As calculated in the previous paragraph p_1 = 150 kPa.

$$p_2 = \frac{p_1 \cdot V_1}{V_2} = \frac{150 \cdot 720}{11683} = 9 \text{ kPa}$$

Now also the impulse for an explosion in the tunnel can be calculated

$$I_s = \frac{1}{2} \cdot P_s \cdot t_p = \frac{1}{2} \cdot 9 \cdot 0{,}200 = 0{,}9 \text{ kPa} \cdot \text{s}$$

Instead of using the increase of volume, the increase of surface area of the wave front can be used to predict the overpressure at the building. A_1 is 72 m^2. According to the figure above A_2 is approximately:

$$A_2 = \left(20 + 14{,}5 + 20\right) \cdot 5 + 14{,}5 \cdot 20 + 20^2 \cdot \tfrac{1}{2} \cdot 2 = 963 \text{ m}^2$$

In correspondence with the method to determine the sound pressure the following applies.

$$p_1 \cdot A_1 = p_2 \cdot A_2$$

$$p_2 = \frac{p_1 \cdot A_1}{A_2} = \frac{700 \cdot 72}{963} = 52 \text{ kPa}$$

3.2.4 Scenario 3: Explosion outside the tunnel on the open road

Five methods that describe the overpressure of an explosion in the open field are discussed here. They are presented in roughly a chronically order of introduction. Only one method will be used.

TNT-equivalence method [10]

This is a method to describe the power of a BLEVE by expressing the explosion in TNT-equivalence. It describes the difference in amount of energy between the liquid and gas state of the substances. This method can only be used at larger distances from the tank (10 to 20 times the tank diameter) and not in tunnels. This TNT-equivalence method is according to [10] very sensitive to its parameters and not really suitable for BLEVE's, because it describes a chemical and not a physical explosion.

The power of the explosion is expressed as an equivalent mass of TNT:

$$Q_{TNT} = \alpha_e \cdot \frac{Q_f \cdot E_{mf}}{E_{mTNT}}$$

Where:

Q_{TNT}	= Equivalent mass of TNT	[kg]
α_e	= TNT equivalency based on energy	[-]
Q_f	= Mass of fuel involved	[kg]
E_{mf}	= Combustion energy of fuel per unit mass	[J/kg]
E_{mTNT}	= Combustion energy of TNT per unit mass	[J/kg]

The dimensionless so-called scaled distance is:

$$\overline{R} = \frac{R}{\left(Q_{TNT}\right)^{1/3}}$$

Where:

R	= Distance to explosion	[m]
$\overline{R}$	= Scaled distance	[-]

The combustion energy of TNT per unit mass is 4,4 MJ/kg [10]. The TNT equivalency α_e can range from 1 to 20 %. For propane a percentage of 3 % is assumed [10]. In literature values from 2 to 10 % can be found. 3 % is most commonly used. The E_{mf} for propane is 49,6 MJ/kg [20].

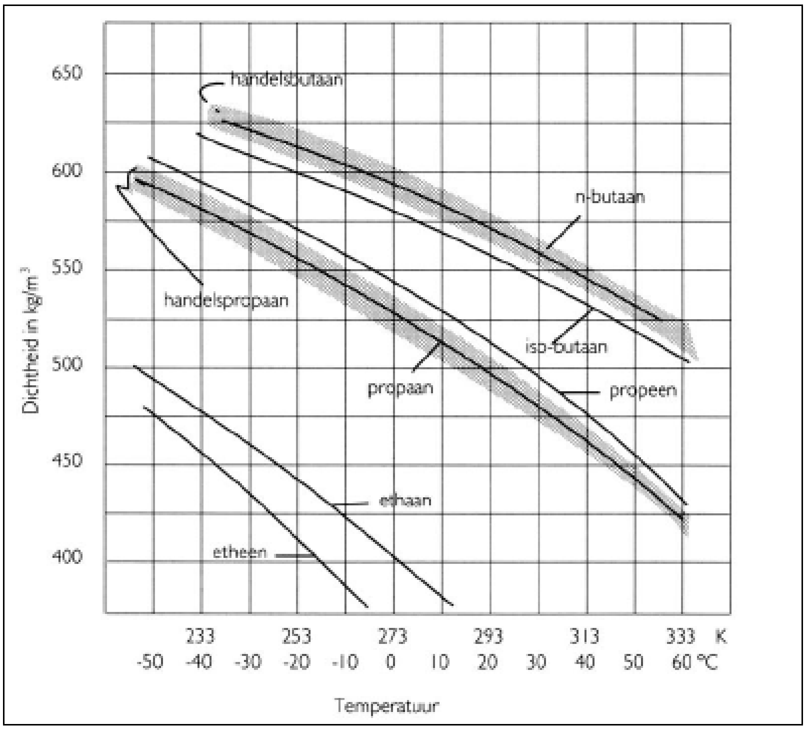


Figure 3-10: Diagram: Density of liquid propane and other hydrocarbons (under vapour pressure) [11]

For a temperature of 326 K the density of liquid propane is:

$$\rho = 435 \text{ kg/m}^3$$

$$V_{tank} = 50 \text{ m}^3$$

$$Q_f = 435 \cdot 50 = 21{,}8 \text{ kg}$$

Now the TNT equivalent mass and the scaled distance can be determined:

$$Q_{TNT} = \alpha_e \cdot \frac{Q_f \cdot E_{mf}}{E_{mTNT}} = 0,03 \cdot \frac{21,8 \cdot 10^3 \cdot 49,6 \cdot 10^6}{4,4 \cdot 10^6} = 7,4 \cdot 10^3 \ kg$$

$$\overline{R} = \frac{R}{(Q_{TNT})^{1/3}} = \frac{20}{(7,4 \cdot 10^3)^{1/3}} = 1,0 \ \mathrm{m/kg}^{1/3}$$

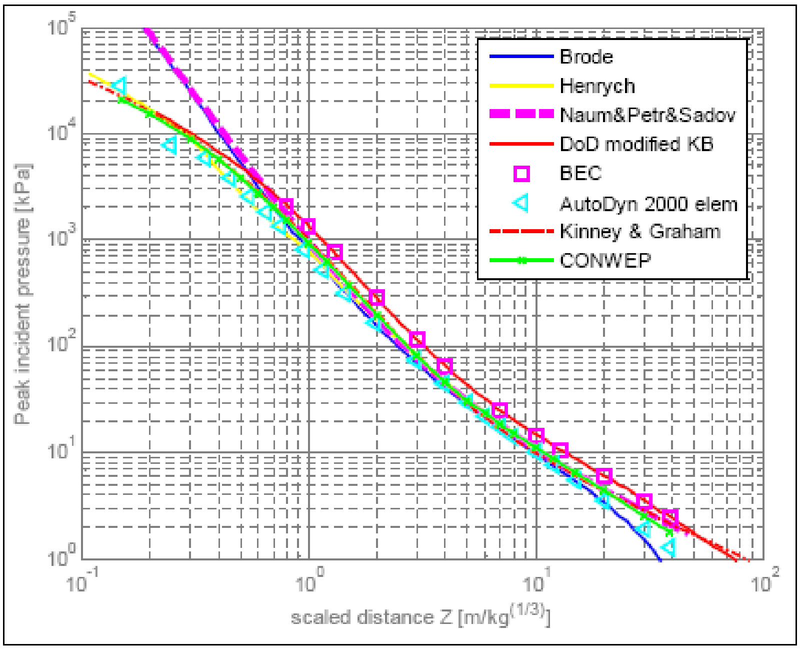


Figure 3-11: Scaled distance (here the symbol Z is used) versus peak overpressure according to several methods [12].

From this diagram it can be read that P_s is 800 to 1350 kPa. The value seems rather large, this can be explained because the TNT-equivalence method does not work well at short distances and it is a very sensitive method; when instead of 3 % TNT equivalency a equivalency of 2% is chosen, the scaled distance becomes $\overline{R} = 1,18$ and P_s will be 500 to 950 kPa. Furthermore this method does not address the phase duration. Because of this sensitiveness, the fact that it is not suitable for BLEVE's and no attention is paid to the impulse this method will not be used.

Diagram method [8]

Based on numerical calculations on the gas dynamics of an expanding sphere of ideal gas a diagram is drawn up from which the blast overpressure can be read. This method is not applicable on a confined space, such as a tunnel. It is based on the TNT-equivalency method, which is used to describe a rule of thumb to determine the expansion energy per unit volume.

Ethaan:
Eex = 0,30*.T – 56* voor 184 K < T < 305 K

Propaan:
Eex = 0,28*.T – 68* voor 231 K < T < 370 K

Butaan:
Eex = 0,25*.T – 68* voor 269 K < T < 425 K

Ammoniak:
Eex = 0,40*.T – 84* voor 240 K < T < 405 K

Kooldioxide:
Eex = 0,33*.T – 22* voor 195 K < T < 304 K

Stikstof:
Eex = 0,39*.T – 32* voor 77 K < T < 126 K

Figure 3-12: Expansion energy E_{ex} [MJ/m^3 liquid] for different substances at certain temperatures T [K], [8].

For propane of 326 K the expansion energy E_{ex} = 23,28 MJ/m^3. The energy of an LPG tank and the scaled distance can now be determined.

$$V_{tank} = 50 \text{ m}^3$$

$$E = E_{ex} \cdot V_{tank} = 23,28 \cdot 50 = 1164 \text{ MJ} = 1164 \cdot 10^6 \text{ Nm}$$

$$\overline{R} = \frac{R}{\sqrt[3]{E/P_0}} = \frac{20}{\sqrt[3]{1164 \cdot 10^6 / 1 \cdot 10^5}} = 0,88 \text{ [-]}$$

The values of the overpressure can be read in the diagram below. The different curves represent different initial overpressures of the explosion.

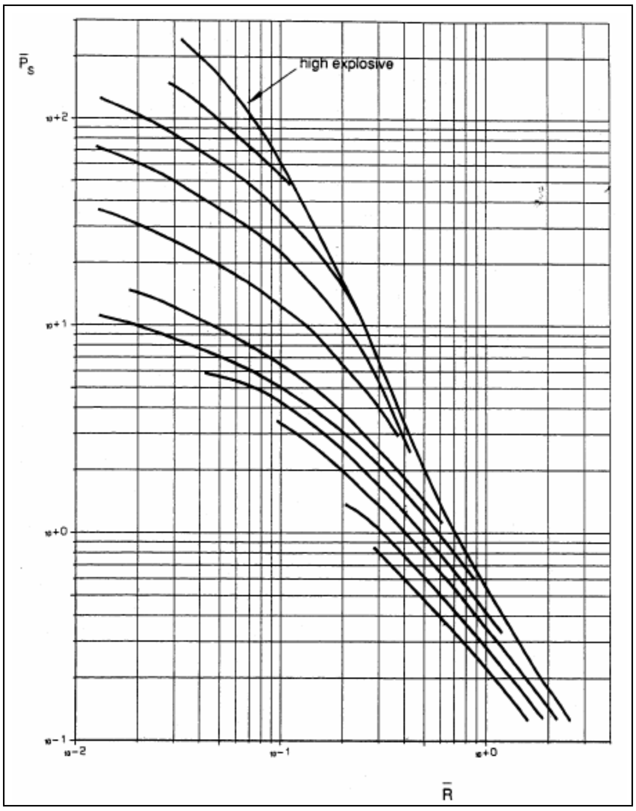


Figure 3-13: Dimensionless overpressure versus dimensionless scaled distance

With the calculated value for the scaled distance and the following formulae the peak overpressure at 20 m form the explosion can be deduced:

$$\overline{P_s} = p_s / p_a - 1$$

$$p_s = \left(\overline{P_s} + 1\right) \cdot p_a$$

$$\overline{P_s} = 0{,}25 \text{ to } 0{,}7$$

$$p_s = 125 \text{ to } 170 \text{ kPa}$$

$$P_s = p_s - P_0 = 25 \text{ to } 70 \text{ kPa}$$

Method from Dynamics reader [6]

With the help of the (conservative) formulas in the reader of the course *dynamics* of the TU Delft it is also possible to determine the overpressure in the open field as a result of an explosion. This method is not suitable for small distances from the explosion.

$$P_s = P_0 \frac{L}{R}$$

$$t_p = 0{,}6 \frac{L}{c_0}$$

$$L = \sqrt[3]{\frac{E}{P_0}}$$

Where:

P_0	= Atmospheric pressure	(=105 N/m^2)
c_0	= Speed of sound in air	(=340 m/s)
R	= Distance from the explosion	[m]
E	= Amount of energy of the explosion	[Nm]

The peak overpressure and positive phase duration are now:

$$L = \sqrt[3]{\frac{E}{p_0}} = \sqrt[3]{\frac{1164 \cdot 10^6}{10^5}} = 22,66 \ m$$

$$p_s = p_0 \frac{L}{R} = 10^5 \frac{22,66}{20} = 113 \ kPa$$

$$t_p = 0,6 \frac{L}{c_0} = 0,6 \frac{22,66}{340} = 40 \ ms$$

So the peak overpressure at a distance of 20 m from the explosion of an LPG tank truck in the open field is 113 kPa. The impulse will now be:

$$I_s = \frac{1}{2} \cdot P_s \cdot t_p = \frac{1}{2} \cdot 113 \cdot 0{,}040 = 2{,}26 \,\text{kPa} \cdot \text{s}$$

This method does not account for the fact that the wave front is faster than the speed of sound. The higher the initial overpressure the higher the speed of the wave front.

Method simple acoustic volume source analogue

In the paper "BLEVE blast by expansion-controlled evaporation" Van den Berg [13] describes a method to calculate the blast effect based on a simple acoustic volume source analogue.

$$\frac{\Delta P}{P_0} = \frac{\gamma}{2\pi \cdot R \cdot c_0^2} \cdot \frac{2 \cdot V \cdot F \cdot \alpha}{(\Delta t)^2}$$

Where:

ΔP	= Blast overpressure	[kPa]
P_0	= Atmospheric pressure	[kPa]
γ	= Ratio specific heats	[-]
R	= Distance from source	[m]
c_0	= Ambient speed of sound	[m/s]
V	= Vessel volume	[m^3]
F	= Flash fraction	[%]
α	= Vapour expansion factor	[-]
Δt	= Release time	[s]

The specific heat ratio is 1,5. The Flash fraction is the percentage of the volume that evaporates explosively, for propane at 326 K it is 50 %. The vapour expansion factor is 260, this means that one cubic metre of liquid will expand in 260 cubic metre of gas.

$$\Delta P = \frac{1,5}{2\pi \cdot 20 \cdot 340^2} \cdot \frac{2 \cdot 50 \cdot 0,5 \cdot 260}{(0,041)^2} \cdot 100 = 80 \text{ kPa}$$

The release time chosen here is t = 41 ms and is deducted from [9] "Pressure load on a cylindrical tunnel caused by an LPG BLEVE: Influence of the vessel failure time" at x=0 and is not the release time in the open field. This method is highly dependent on the failure time of the tank.

Gas dynamic modelling

Also in the paper "BLEVE blast by expansion-controlled evaporation" [13] a gas dynamic modelling for a BLEVE blast in the open field for instantaneous vessel rupture is presented. It gives the following log-log graphs from which the peak overpressure and positive phase duration can be read.

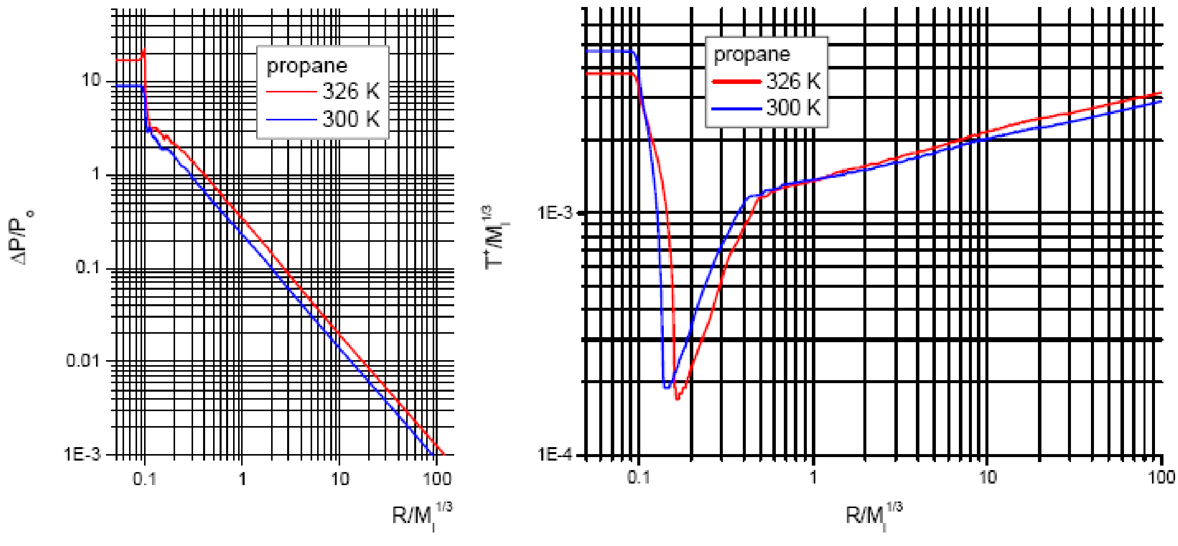


Figure 2 *BLEVE blast charts for propane, flashing from 326 K and 300 K respectively*
ΔP = overpressure (kPa); *P_o = ambient pressure (kPa);*
T^+ = positive phase duration (s); *M_l = liquid mass (kg);*
R = distance from source centre (m)

Figure 3-14: BLEVE blast charts for propane

$$\frac{R}{M_l^{1/3}} = \frac{20}{(50 \cdot 509)^{1/3}} = 0{,}68\ \mathrm{m/kg}^{1/3}$$

The paper uses a density of 509 kg/m^3. Earlier in this report a density for propane of 435 kg/m^3 is used, which would result in a value of 0,72 for the previous equation. A density of 509 kg/m^3 is used here because it is used in the paper.

From the graph can now be read:

$$\frac{\Delta P}{P_0} = 0{,}52$$

$$\Delta P = 100 \cdot 0{,}52 = 52\ \mathrm{kPa}$$

$$\frac{\mathrm{T}^+}{M_l^{1/3}} = 0{,}00136\ \mathrm{s/kg}^{1/3}$$

$$T^+ = 0{,}00136 \cdot (50 \cdot 509)^{1/3} = 40\ \mathrm{ms}$$

Now the positive phase duration (expressed in this paper as T+ and not t_p)

$$I_s = \frac{1}{2} \cdot P_s \cdot t_p = \frac{1}{2} \cdot 52 \cdot 0{,}040 = 1{,}04\ \mathrm{kPa \cdot s}$$

3.3 Design load

3.3.1 Choice of scenario and method

The peak overpressure for three scenarios and a total of nine methods is calculated and gathered in the table below.

	Method	Initial over pressure (kPa)	Over pressure at tunnel mouth (kPa)	Side on over pressure at building P_s (kPa)	Positive phase duration t_p (ms)	Impulse I_s (kPa*s)
Explosion in the tunnel	Shock-tube-formula	450	-	-	-	-
	Method Van de Berg	1280	150	9	200*	0,9
Explosion in the tunnel mouth	Volume increase	-	700*	43	-	-
	Surface increase	-	700*	52	-	-
Explosion on the open road	TNT-equivalence method	-	-	800-1350	-	-
	Diagram method	-	-	25-70	-	-
	Reader dynamics	-	-	113	40	2,26
	Acoustic volume source analogue	-	-	80	-	-
	Gas dynamic modelling	-	-	52	40	1,04

Table 3-3: Overview blast load methods

* Assumptions author

The Shock-tube-formula method gives an initial overpressure of 450 kPa. However this formula is based on pure gas dynamics of a bursting vessel and therefore does not apply to a BLEVE in a tunnel. Furthermore with this method no attention is paid to the duration of the load and therefore nothing can be said about the impulse on the structure. The duration of the load depends on the volume of the tank. The method of Van de Berg does give a time duration of the load and is specially set up for a BLEVE in a tunnel. Therefore for the initial overpressure of an explosion in a tunnel the method of Van de Berg is used. At the location of the explosion the positive phase is 41 ms. In the absence of other data a positive phase is assumed near the building of 200 ms and an overpressure of 150 kPa remains at the tunnel mouth. As calculated this results in a peak overpressure of 9 kPa and an impulse of 0,9 kPa*s.

The side on overpressure P_s on a building as a result of an explosion in the tunnel mouth is 43 kPa according to the volume increase method. This method is very sensitive for which values are chosen for the initial overpressure in the tunnel mouth and the volumes V_1 and V_2. These values are rough estimations and therefore this method is not very reliable. If instead of 700 kPa for the initial overpressure and 10 m for volume V_1, 1000 kPa and 20 m is chosen, which are reasonable values, then the side on overpressure at the building will be 123 kPa. The overpressure is 52 kPa with the surface increase method. This method is also very sensitive. Both methods don't give a good estimation for the impulse on the building.

For an explosion in the open field five methods have been described. They all conclude to a different value for the side on overpressure on the building. With the TNT-equivalence method a value for P_s of 800 to 1350 kPa is found. The diagram method gives a value of 25 to 70 kPa. With the help of the reader of *dynamics* a value of 113 kPa is found. The simple acoustic volume source method gives a value of 80 kPa and a gas dynamic modelling says P_s = 52 kPa. Of the methods that determine the overpressure as a result of an explosion in the open field only the method from the reader *Dynamics* and the Gas dynamic modelling method give a value for the impulse, 2,26 kPa*s and 1,04 kPa*s respectively. The values for P_s differ very much. The different methods are plotted below.

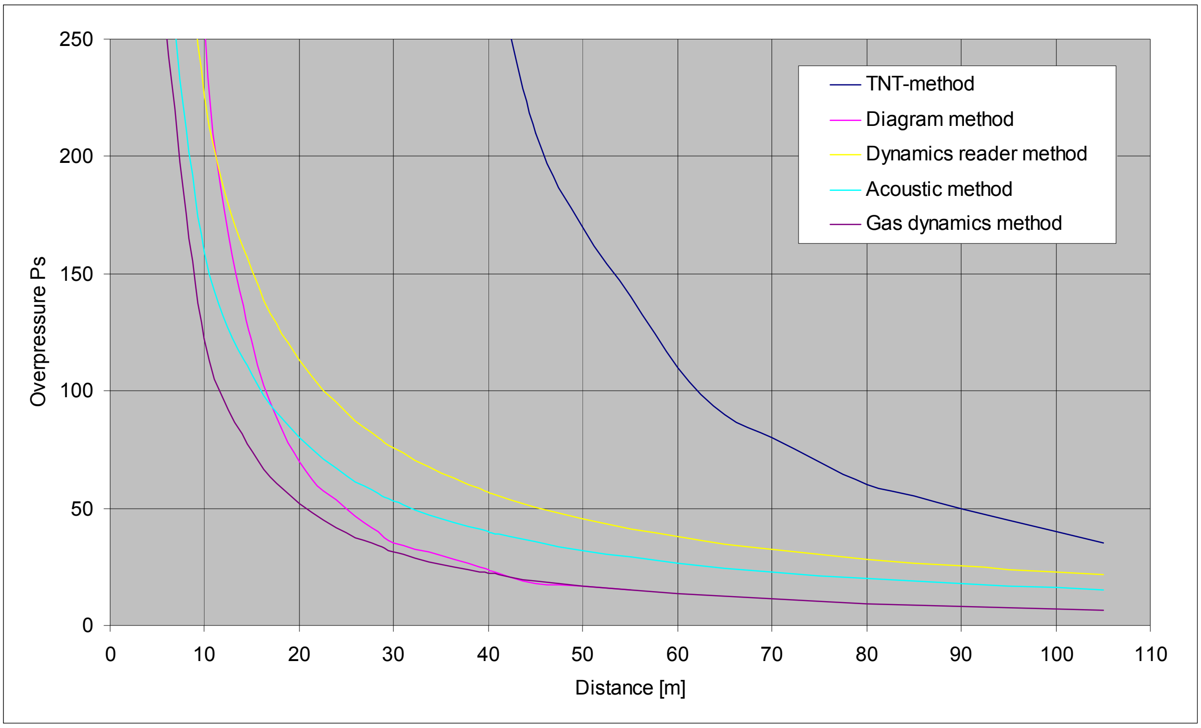


Figure 3-15: Pressure-distance diagram for five different methods

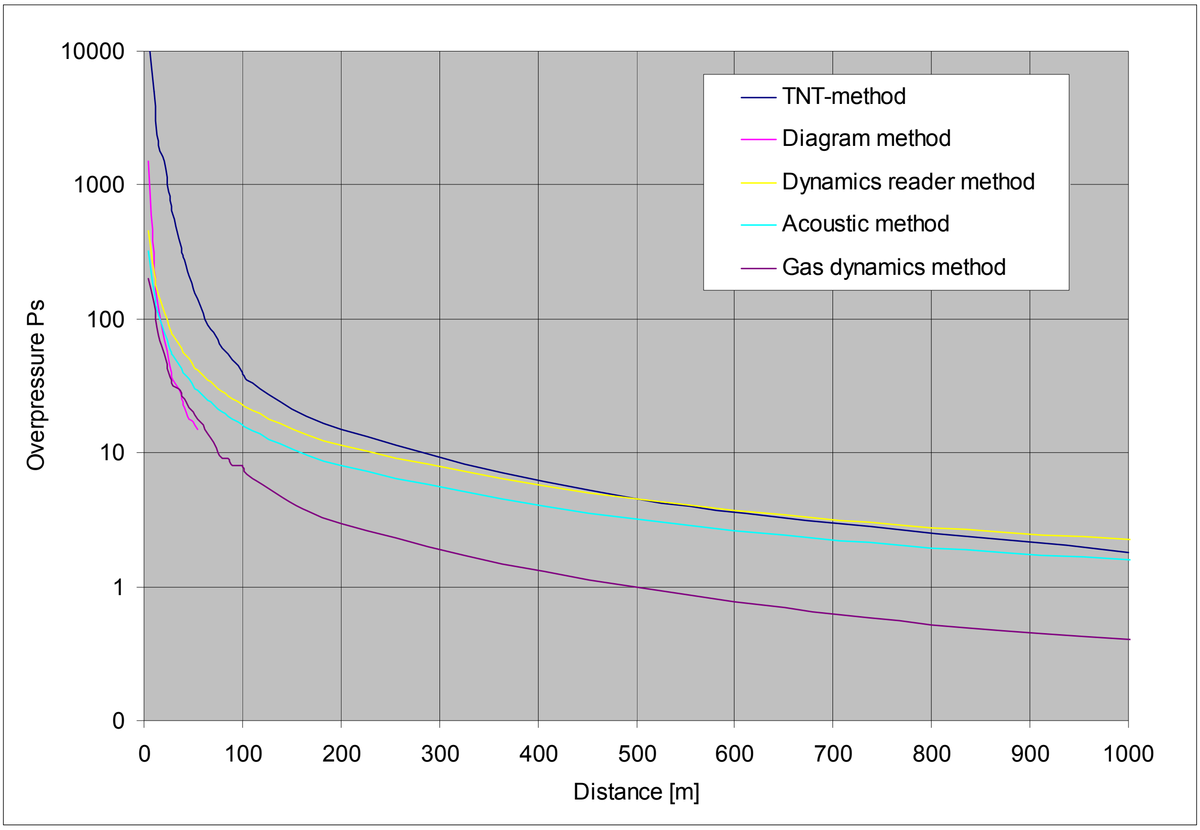


Figure 3-16: Pressure-distance diagram for five different methods

The methods discussed were presented in a chronically order, the TNT-method being the oldest. The newer the method the more insight in the theory of explosion dynamics it represents. The TNT method doesn't consider the duration of the load and is not applicable for BLEVE's. The diagram method and dynamics reader method are very conservative. Plus the diagram method is not applicable to distances larger than 55 m. The last method, the gas dynamics method, is preferred over the acoustic method because it also gives a tool to determine the positive phase duration, so this method will be used.

An estimation of the impulse is required so only the method Van de Berg and the Gas dynamic modelling method remain. The latter is preferred over the first because the values for P_s and I_s are larger. In conclusion: the side on overpressure at a building at 20 m from an explosion of an LPG tank of 50 m^2 is 52 kPa with a positive phase duration of 40 ms and an impulse of 1,04 kPa*s.

An explosion on the open road gives a larger side on overpressure at the building in comparison with an explosion deep in the tunnel. With an explosion in a tunnel a lot of energy is lost by bending the pressure waves in the two directions of the tunnel. An explosion in the tunnel mouth results in an equal or a slightly lower peak overpressure than an explosion on the open road. Despite that an explosion in the tunnel or the tunnel mouth has a larger initial overpressure than an explosion on the open road, the side on overpressure at the building is higher as a result of an explosion on the open road. An explanation for this is that the distance of the building from the explosion in the tunnel or the tunnel mouth is larger than an explosion on the open road in front of the building.

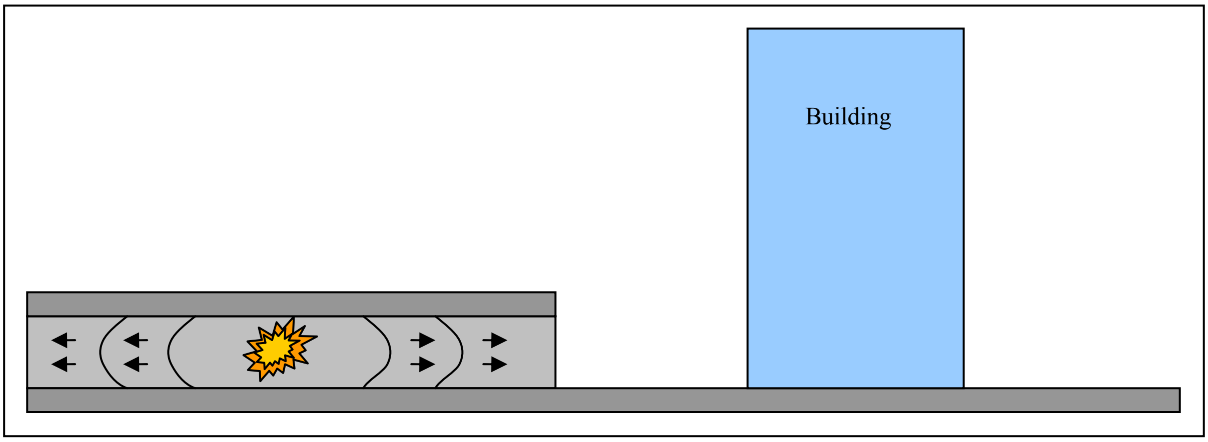


Figure 3-17: Explosion in tunnel

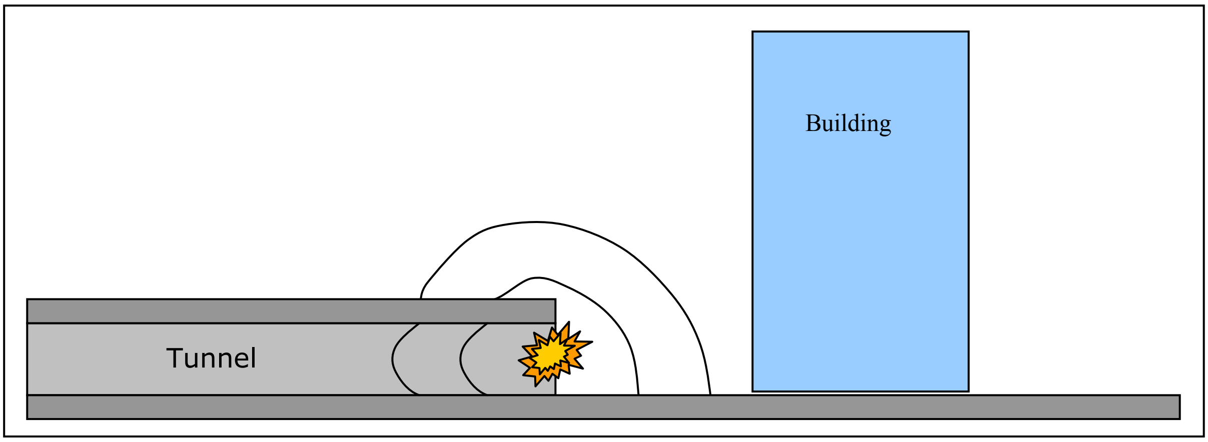


Figure 3-18: Explosion in tunnel mouth

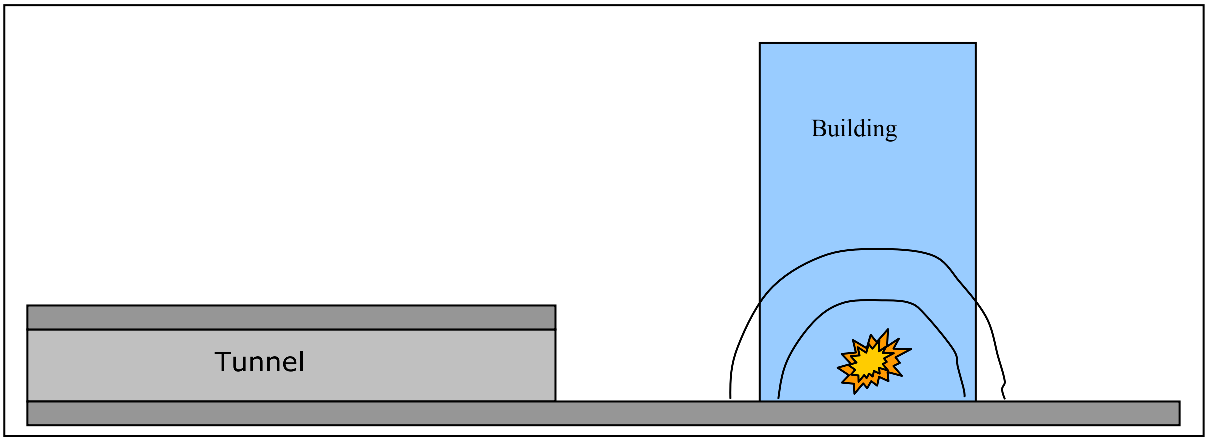


Figure 3-19: explosion on the open road

So the scenario of an explosion on the open road is governing and therefore determines the load on the building that will be worked with in the rest of this study. The hypothesis that an explosion in the tunnel or in the tunnel mouth would result in a larger overpressure at the building than an explosion on the open road turns out to be not true. On basis of the used methods it can be concluded that an explosion on the open road gives a larger blast load on the building than an explosion in the tunnel or the tunnel mouth.

Scenario	Method	Side on over pressure at building P_s (kPa)	Positive phase duration t_p (ms)	Impulse I_s (kPa*s)
3	Gas dynamic modelling	52	40	1,04

Table 3-4: Design load

3.3.2 Sensitivity analysis

By changing the parameters that determine the peak side on overpressure and the positive phase duration there values can be changed. These parameters are:

- Distance from the source
- Tank volume
- Liquid density
- Atmospheric pressure

By doubling the distance to the explosion from 20 m to 40 m the peak overpressure drops to 23 kPa and the positive phase duration increases slightly to 45 ms. By decreasing the tank volume from 50 to 20 m^3 the peak overpressure changes from 52 to 36 kPa and t_p drops to 31 ms. If it would be possible to have a lower atmospheric pressure than 100 kPa, for example 90 kPa, then the overpressure will be 47 kPa instead of 52 kPa.

This sensitivity analysis shows that considerable reductions can be reached for the overpressure at the building by changing the parameters, especially the distance. In the aim of this study to give a blast-resistant building design a reduction of the blast load is desirable and therefore a larger distance from the source and smaller tank would be a means to make the building resistant to an explosion of an LPG tank truck. On the other hand these solutions are evident.

4 Building response

4.1 Introduction

The blast-wave of an explosion of an LPG tank truck can cause damage to the surroundings. Besides the gravity of the explosion the blast load on the buildings along the road depends on several other aspects, such as reflection and the shape of the building. Several aspects and how they determine the load of the blast-wave that is transferred to the main load bearing structure are discussed in the second paragraph of this chapter. After this the properties of a building that determine its response to this load are determined. When the load, all the response aspects and the structural properties are known an estimation of the damage to the building can be given.

In chapter 5 a model building is defined and the values of the load and the dimensions of the model building will be used to determine the response of the building with the methods described in this chapter.

4.2 Response aspects

What are the effects of the shape and the material of the façade, the building as a whole, the structural elements, the windows, etc on the load of the explosion that will be transferred to the main load bearing structure? The different aspects that play a role are divided into paragraphs. For each aspect the available methods are described.

4.2.1 Reflection

A blast-wave obstructed by an obstacle will result in a disturbed behaviour of the blast-wave. When the pressure wave hits the building it will reflect against it. This results in a larger load on the building than the incoming peak overpressure. The ratio between the overpressure of the reflected wave and the incoming wave is called the reflection coefficient or short rc. This coefficient depends on the angle on which it hits the building, the peak overpressure and the type of wave.

$$rc = \frac{P_r}{P_s}$$

Where:

P_r	= Reflected peak overpressure	[kPa]
P_s	= Incoming peak overpressure	[kPa]

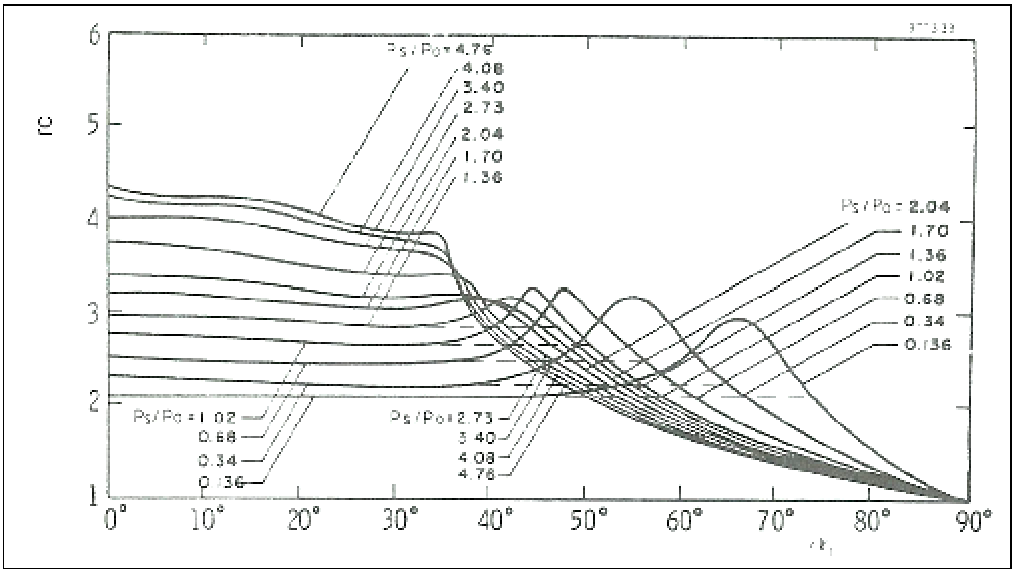


Figure 4-1: rc for shockwaves for different angles and pressures. The experimentally found increase of rc for angles between 40 and 75 degrees can not be explained and therefore the dotted line is used instead [7].

The rc is at its maximum for an angle of incidence of 0° and is one for an angle of 90°. The higher the overpressure of the shockwave the larger the rc.

Changing the geometry of the building from square to round affects the reflection coefficient, because the average angle of incidence for a round building is higher. A reduction of the overpressure on the building can be obtained if the building has a higher angle with the incoming pressure wave.

The perpendicularly reflected overpressure is also given by the following formula.

$$P_r = 2 \cdot P_s + \frac{(\gamma + 1) \cdot P_s^2}{(\gamma - 1) \cdot P_s + 2 \cdot \gamma \cdot P_0}$$

Where:

P_r	= Reflected overpressure	[kPa]
P_s	= Peak overpressure	[kPa]
γ	= C_p / C_v	[-]
C_p	= The specific heat with constant pressure	[J/(kg*K)]
C_v	= The specific heat with constant volume	[J/(kg*K)]
P_0	= Atmospheric pressure	[kPa]

For air with a γ of 1,4 the formula becomes:

$$P_r = 2 \cdot P_s + \frac{2{,}4 \cdot P_s^2}{0{,}4 \cdot P_s + 2{,}8 \cdot P_0}$$

4.2.2 Dynamic blast load

Besides a pressure increase there is also a displacement of air which results in a force on the building [7]. A formula for this is:

$$Q = \frac{5}{2} \cdot \frac{P_s^2}{\left(7 \cdot P_0 + P_s\right)}$$

$$Q_D = C_D \cdot Q$$

Where:

Q	= Air displacement pressure	[kPa]
P_s	= Peak overpressure	[kPa]
P_0	= Atmospheric pressure	[kPa]
Q_D	= Dynamic blast load	[kPa]
C_D	= Drag coefficient	[-]

C_D is the drag-coefficient which depends on the shape of the structure. For side on reflection on cube-like structures C_D = 1,05.

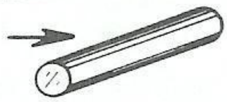
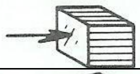
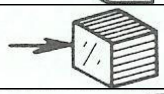
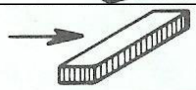
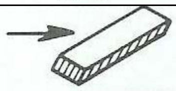
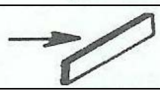
Shape	Figure	C_D
Long straight cylinder		1,2
Sphere		0,47
Cylinder		0,82
Disk		1,17
Cube		1,05
Cube		0,80
Elongated box		2,05
Elongated box		1,55
strip		1,98

Table 4-1: C_D Coefficient

4.2.3 Pressure relief wave

Besides pure reflection and dynamic blast load other phenomena happen when a blast-wave travels around a building. The behaviour of the blast-wave around a structure is presented in the figures below.

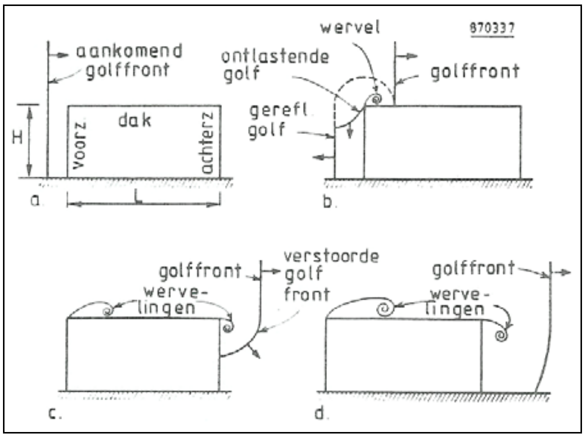


Figure 4-2: Schematic reproduction of the disturbance of the blast-wave around a structure, a side view [7].

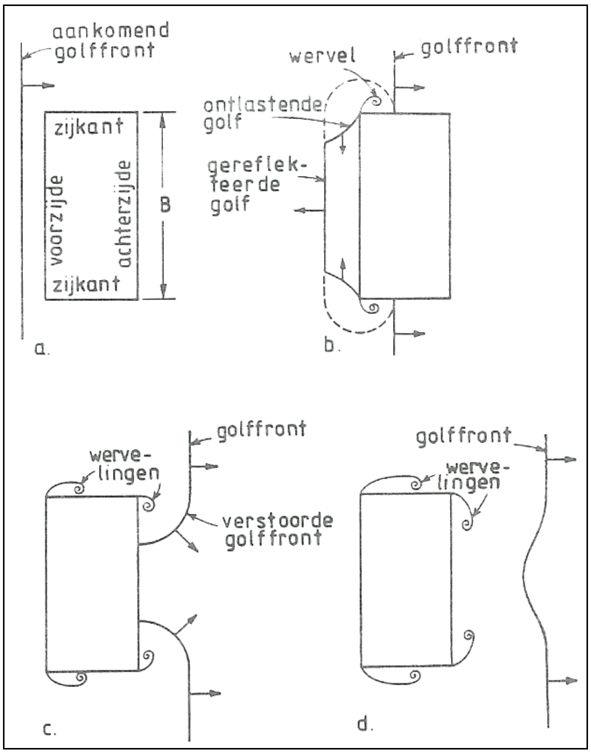


Figure 4-3: Schematic reproduction of the disturbance of the blast-wave around a structure, a top view [7].

Because of disturbance of the pressure wave at the edges of the building a relieving pressure wave travels along the front of the building from the sides and the top. The pressure will not get lower than the incoming wave plus the dynamic load. The time t_s in which the reflected overpressure is equal to the incoming overpressure plus the dynamic pressure is:

$$t_s = \frac{3S}{U}$$

$$U = c_0 \sqrt{1 + \frac{6P_s}{7P_0}}$$

Where:

S	= Smallest value of H and ½*B	[m]
H	= Height of the building	[m]
B	= Width of the building	[m]
U	= Speed of wave front	[m/s]
c_0	= Speed of sound at atmospheric pressure	(=+340 m/s)

This method shows that also the height and width of the building determine the pressure load on the building.

The horizontal load against time as a result of the overpressure, the reflection and the dynamic load can now be captured in the following diagram.

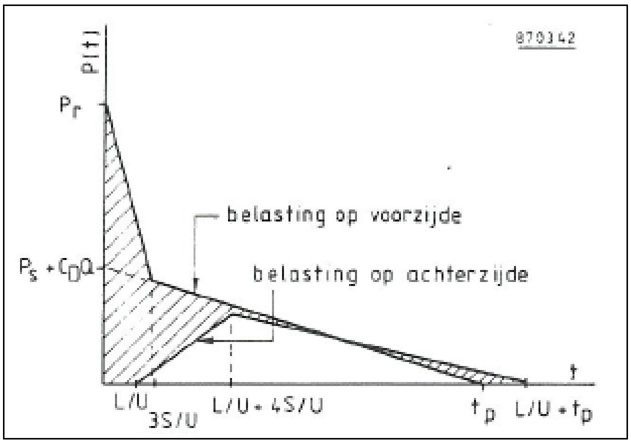


Figure 4-4: Schematic reproduction of the pressure-time curve at a finite reflecting surface [7].

As the graph shows, the impulse will be different from the schematized pressure-time lapse of paragraph 3.2.1 due to the reflected overpressure and dynamic blast load. The impulse can be calculated by determining the surface under the graph.

$$I_s = \frac{1}{2} \cdot t_s \cdot (P_r - P_s - Q_D) + \frac{1}{2} \cdot t_p \cdot (P_s + Q_D)$$

4.2.4 Adjacent buildings

The structure and shape of the city at the location of the explosion can have considerable influence on the overpressure near the buildings. In urban environments where the buildings are high or close to each other the pressure as a result of an explosion will be higher than in cities with low buildings and wide streets or even an open field. The situation of an infinite row of high-rise buildings will react very differently than just a few large buildings with open space in between where there is much more room for pressure release. Just as a dense building geometry can enhance the overpressure it can also be reduced by shadowing.

Experimental investigation [14] on a T-junction with and without buildings along the road show a large difference, see diagram. Another numerical and experimental study by Rose and Smith [Influence of the principal geometrical parameters of straight city streets on positive and negative phase blast wave impulses] showed that also de street width and building height are of importance. The outcome was that a significant enhancement of the impulse and overpressure occur depending on the city geometry.

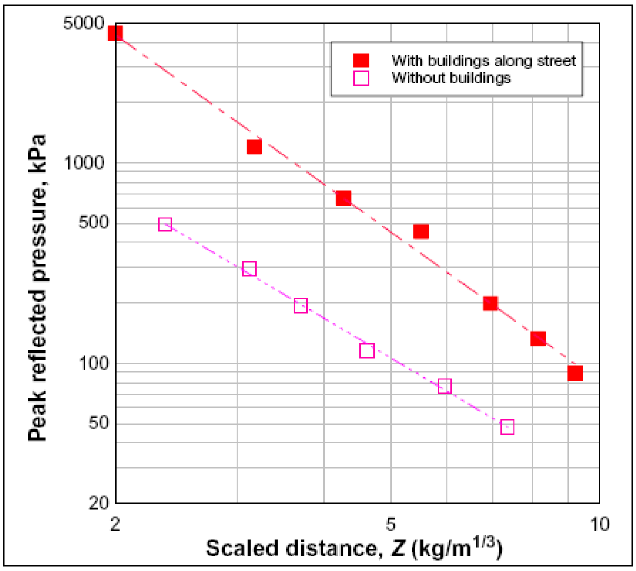


Figure 4-5: Experimental peak reflected overpressure on target building at the end of a street with and without adjacent buildings along the road. This diagram shows that the reflected overpressure can be a factor 3 to 6 higher with buildings along the road than the free field pressure [14].

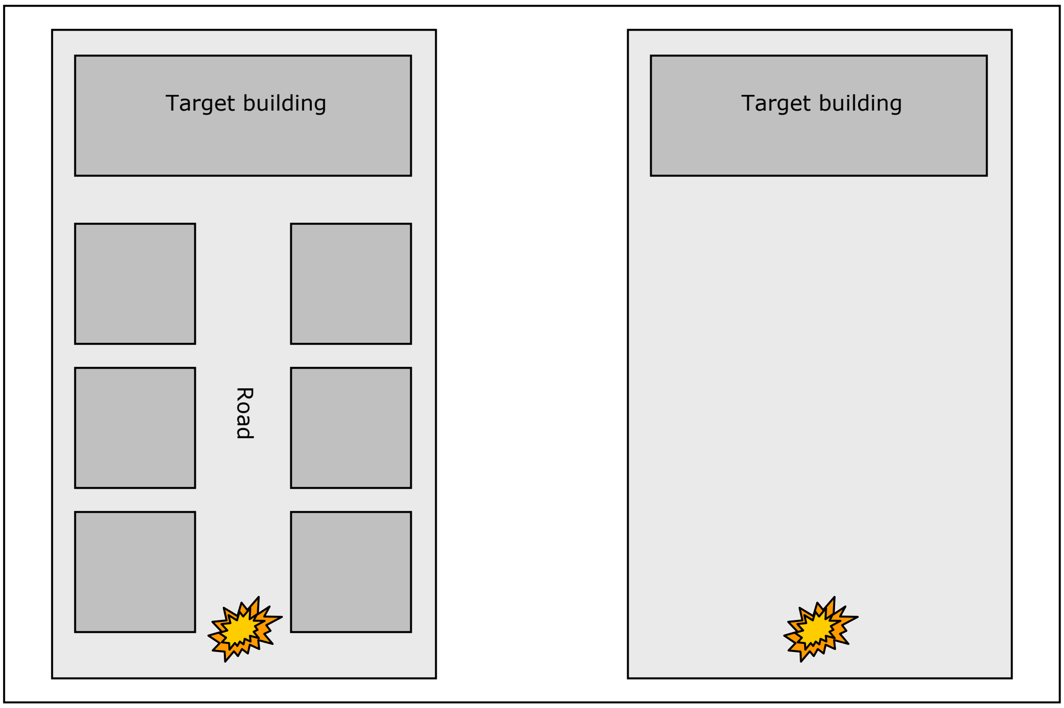


Figure 4-6: Experiment setup with and without adjacent buildings [14]

Some software tools have been developed to determine the course of the overpressure in a 3D environment. A simulation of such a tool [14] for 25 ms takes a supercomputer 160 hours to compute and is therefore not very practical. In the study [14] a model with two buildings behind each other was simulated. An explosion in front of the first building with a building behind it resulted in a factor 3 higher reflected overpressure and impulse at the rear of the first building than with the second building not present. These results confirm that the geometry of the buildings have a large influence.

According to [15, 16] highly densely build streets in urban areas have an impulse enhanced by a factor five compared with side on values at equivalent distances. Also the experiments show that the negative phase of the pressure wave can have a considerable attribution to the load.

4.2.5 Distribution blast on façade of building

In the event of an explosion on the road the blast waves will act on the buildings beside the road. The top of the building has a larger distance and angle to the source than the foot of the building. Also a larger horizontal distance parallel to the road will result in a larger impact angle. The side on overpressure on the building will be smaller for larger distances. A larger impact angle will result in a smaller reflection coefficient and thus a smaller reflected peak overpressure. Different distances and impact angles will result in different reflected peak overpressure for every point on the façade on the building beside the road.

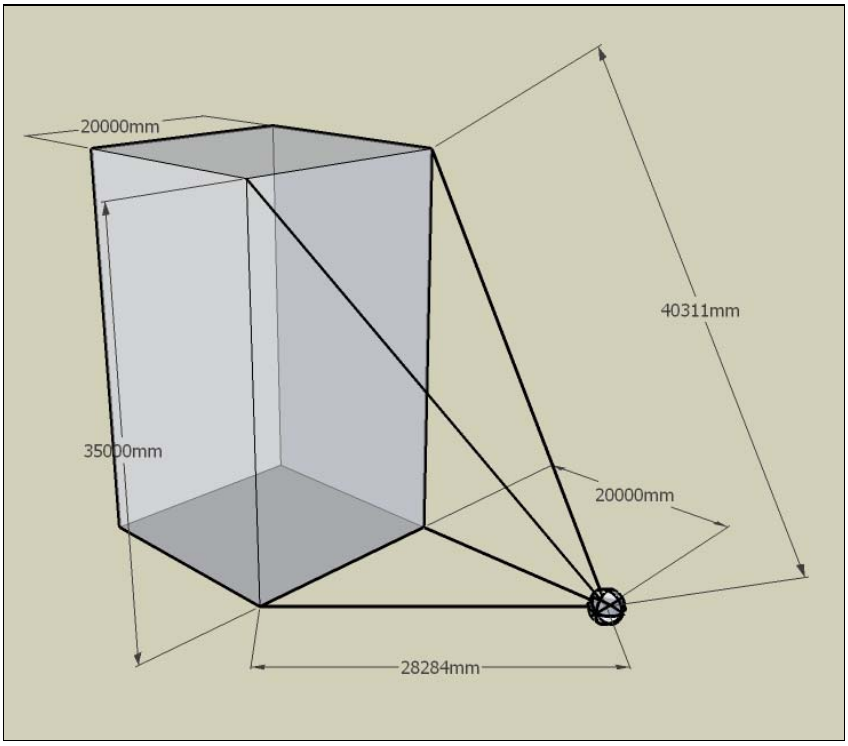


Figure 4-7: Different distances and angles for all points on the facade

4.2.6 Glass failure

Because glass is very brittle it will break easily [7]. In general glass will be the first part of the building to fail. When the pressure wave hits the building the pressure wave will reflect creating the peak reflected overpressure. But if the glass fails instantly this pressure built-up on the windows will not occur. The overpressure will be lower when the windows break, which results in a lower load that will be transferred to the main load bearing structure. How much this reduction is depends on how fast the windows will fail, the percentage of windows in the façade and the dimensions and material properties of the glass.

If the glass fails very easily or in other words the failure pressure and the failure time are very low the reduction of the reflected overpressure will be high. If the failure time of the glass is more than the positive phase duration of the load the glass will not fail. When the glass does not fail the reflected overpressure will be the peak reflected overpressure, just as on the rest of the building.

The questions that need to be answered are: At what overpressure does glass fail and how fast does it fail. Because glass is very brittle it is assumed that it has a failure time of zero; the glass will fail immediately. This means the impulse is zero. The overpressure on the glass is the failure strength of the glass.

Because it is assumed that the glass fails instantly the building can be considered as a block with many holes instead of a solid block. The peak reflected overpressure and the impulse on the whole façade is lower because of the presence of glass in the façade.

$$P_r' = \frac{1}{A_{total}}\left(\left(A_{total} - A_{glass}\right)\cdot P_r + P_{st,glass}\cdot A_{glass}\right)$$

$$I_s' = \frac{A_{total} - A_{glass}}{A_{total}}\cdot I_s$$

Where:

P'_r	= Corrected peak reflected overpressure	[kPa]
A_{total}	= Total surface of facade	[m^2]
A_{glass}	= Surface of glass in façade	[m^2]
P_r	= Peak reflected overpressure	[kPa]
$P_{st,glass}$	= Static strength glass	[kPa]
I'_s	= Corrected impulse	[kPa*s]
I_s	= Impulse	[kPa*s]

This corrected peak reflected overpressure and impulse is the average on the surface of the building with the failure of glass taken into account. This means that if a building has an entirely glass façade the overpressure will be the failure strength of the glass and the impulse will be zero. However the entering of the pressure wave and flying around glass causes serious danger to the people inside. A building that has a closed facade with no windows will have a reflected peak overpressure P_r as calculated in paragraph 4.2.1 t/m 4.2.3.

4.2.7 Structural elements

When the pressure load hits the building the load imposed is transferred from the façade element to the columns, beams, walls and floors of the building. How this load is transferred and thus how much load each element endures, depends on the specific design of the structure. Al separate elements can fail under the imposed load. This structural response of each element depends on the reflected peak overpressure, the impulse, the material properties and dimensions and the location of the element in the building.

It is assumed that the façade elements will not fail or at least transfer the pressure load to the elements behind it. The load on the façade elements is not transferred to separate beams and columns in the façade; the façade elements transfer the load to the floors behind it. The beams of the structure that support the floor are considered to be horizontally supported by the floor; the beams and the floor act as one system in the horizontal direction. Because of the large dimension of the floor in comparison with the beams and columns it has a large strength and stiffness in this direction. The load on the floors is the overpressure times the surface area of the façade which transfers its load to the floor, which is the storey height. So the overpressure on the building acts as an evenly distributed line load on one side of the floor.

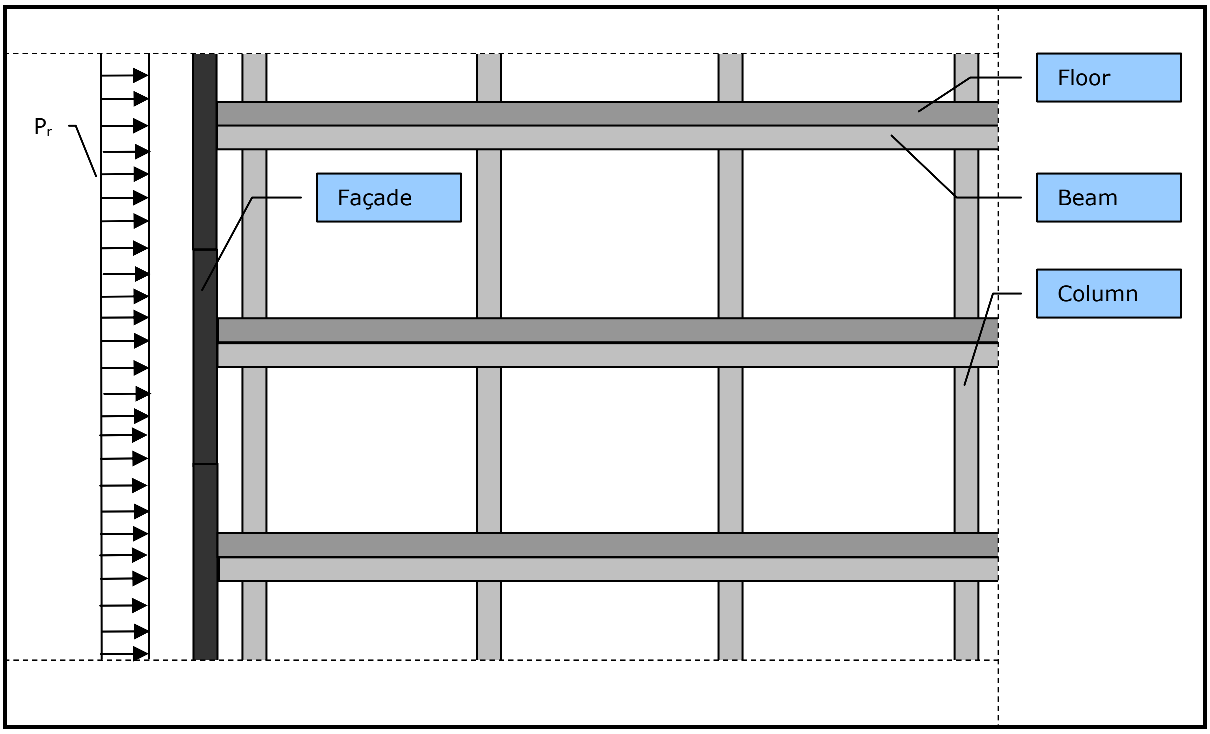


Figure 4-8: Cross section of part of building

When the blast wave enters the building (because of the failure of glass) it can act on separate building elements such as floors, columns and walls. When columns are used in the façade the load on the façade elements is transferred directly to the column. The strength of a column in this direction will be smaller than a floor or wall.

The floors act as diaphragms and transfer the load to the stabilizing elements such as walls and cores.

The response of the elements is regarded to be the same as the response of the whole building. The exact load on the elements will be determined in chapter 5 when the model building is defined. The mechanical properties of the elements will be determined in paragraph 4.3, then it will be clear whether the elements will fail or not under the imposed load. This local behaviour of the structure can possibly be governing.

Note:
If an element does not fail instantly or very slow, then energy of the explosion is lost in deformation of the element, just as in the case of glass failure, which reduces the load on the main load bearing structure. The speed of failure is essential and unknown. Further study is recommended.

4.2.8 Type of main load bearing structure

The design of the main load bearing structure can be a single concrete core, tube in tube (with a load bearing façade) or multiple cores and walls. Also a trussed frame and suspending structures can be used. And there are countless other variations of main load bearing structures. What the effect is of the design of the main load bearing structure on the load that the building endures and how it will respond to this load is unknown and recommend for further study.

4.3 Mechanical properties building

The deformation and internal stresses of a building depend on the load, how the different elements of a building are composed together and the material and dimensions of the components. Because of the very short duration of the load (40 ms) it is called a dynamic load. The strength of the structure determines if it can withstand the explosion load. Because the load changes in time the building can vibrate in its natural frequency. The imposed explosion load is far larger than for which most buildings are designed. So a certain amount of damage can be expected. The ability of a building to withstand this excessive loading can be expressed in the maximum permanent deformation, which is also called the ductility.

The parameters that are of importance for the structural response of the building are:

For the load
- The shape; shock wave or pressure wave
- Peak overpressure
- Positive phase duration

For the structure
- Static strength
- Natural frequency
- Ductility

The parameters for the load are determined in the chapter 3. This paragraph gives methods to determine how much load a building can take. Subsequently the three structure characteristics depicted above, static strength, natural frequency and ductility are worked out. As will turn out several mechanical properties of the building have to be known. Also the properties of structural elements are discussed.

4.3.1 Static strength

The static strength of a building is the ultimate strength which is the ability of the building to resist a horizontal working load on the façade of the building without collapse of the building, but with permanent deformations. Any increase of the load above the static strength will result in the collapse of the building; the ultimate limit state. If only cracks and deformations occur but the building will not collapse it is called the serviceability limit state.

For the calculation of structures the loads are multiplied by a safety factor. For the variable wind load in the ultimate limit state on a concrete office building this factor is 1,5, see the table below. The calculated stresses in the structure have to meet a required value; this is however the lower limit of the stresses and not the average stress. The difference is a factor 1,5 [7]. A load is higher when the loading is very short, which is the case, therefore another factor 1,2 [7] can be added to the safety. The total safety factor β is now 2,7 for concrete. Steel has a safety factor of 2,0 [7]. Because an explosion is a special load it is only required that the building does not collapse. Therefore no safety factor is required and the static strength can be multiplied by this factor β.

	Veiligheids-klasse	Belastings-combinatie volgens 6.3.4	$\gamma_{f;g;u}$ ($\gamma_{f;p;u}$)		$\gamma_{f;q;u}$	$\gamma_{f;a;u}$
			normaal (ongunstig)	gunstig		
Fundamentele combinaties	1 2 3 1-2-3	(1) (1) (1) (2)	1,2 1,2 1,2 1,35 [a]	0,9 0,9 0,9 0,9	1,2 1,3 1,5 –	– – – –
Bijzondere combinaties	1-2-3	(3)	1,0	1,0	1,0	1,0

[a] Indien de permanente belasting één oorzaak heeft, mag in combinatie met gunstig werkende belasting de waarde van 1,35 zijn gereduceerd tot 1,2.

Figure 4-9: Load factors ultimate limit state [21]

To determine the static strength of high buildings the governing horizontal design load is the wind load. The wind load is according to the code NEN 6702 [21] dependent on the height and the location of the building.

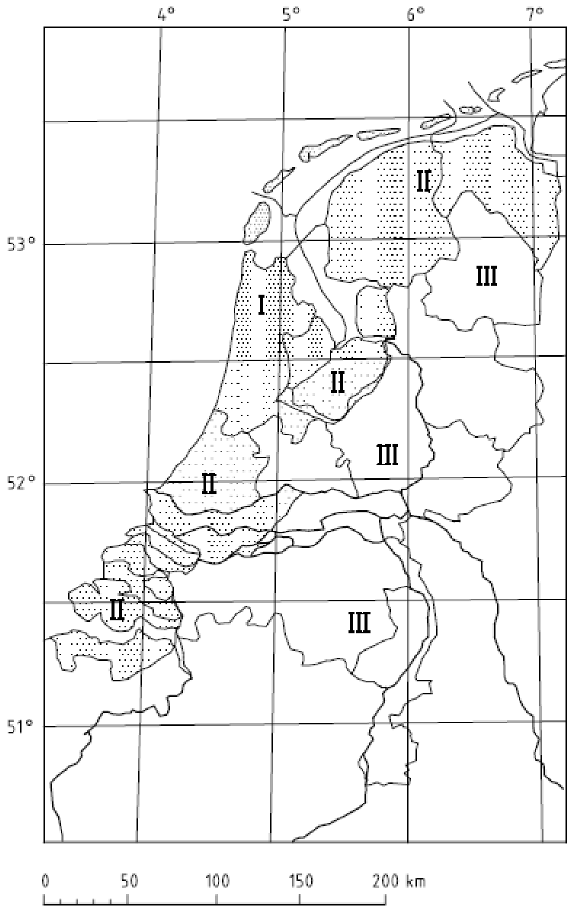


Figure 4-10: Subdivision of the Netherlands in three areas according to the wind pressure [21]

h m	p_w kN/m²					
	Gebied I		Gebied II		Gebied III	
	onbebouwd	bebouwd	onbebouwd	bebouwd	onbebouwd	bebouwd
≤ 2	0,64	0,64	0,54	0,54	0,46	0,46
3	0,70	0,64	0,54	0,54	0,46	0,46
4	0,78	0,64	0,62	0,54	0,49	0,46
5	0,84	0,64	0,68	0,54	0,55	0,46
6	0,90	0,64	0,73	0,54	0,59	0,46
7	0,95	0,64	0,78	0,54	0,63	0,46
8	0,99	0,64	0,81	0,54	0,67	0,46
9	1,02	0,64	0,85	0,54	0,70	0,46
10	1,06	0,70	0,88	0,59	0,73	0,50
11	1,09	0,76	0,91	0,64	0,76	0,54
12	1,12	0,81	0,94	0,68	0,78	0,58
13	1,14	0,86	0,96	0,72	0,80	0,61
14	1,17	0,90	0,99	0,76	0,82	0,64
15	1,19	0,94	1,01	0,79	0,84	0,67
16	1,21	0,98	1,03	0,82	0,86	0,70
17	1,23	1,02	1,05	0,85	0,88	0,72
18	1,25	1,05	1,07	0,88	0,90	0,75
19	1,27	1,08	1,09	0,90	0,91	0,77
20	1,29	1,11	1,10	0,93	0,93	0,79
25	1,37	1,23	1,18	1,03	1,00	0,88
30	1,43	1,34	1,24	1,12	1,06	0,95
35	1,49	1,43	1,30	1,20	1,11	1,02
40	1,54	1,50	1,35	1,26	1,15	1,07
45	1,58	1,57	1,39	1,32	1,19	1,12
50	1,62	1,62	1,43	1,37	1,23	1,16
55	1,66	1,66	1,46	1,42	1,26	1,20
60	1,69	1,69	1,50	1,46	1,29	1,24
65	1,73	1,73	1,53	1,50	1,32	1,27
70	1,76	1,76	1,56	1,54	1,34	1,31
75	1,78	1,78	1,58	1,57	1,37	1,33
80	1,81	1,81	1,61	1,60	1,39	1,36
85	1,83	1,83	1,63	1,63	1,41	1,39
90	1,86	1,86	1,65	1,65	1,43	1,41
95	1,88	1,88	1,68	1,68	1,45	1,44
100	1,90	1,90	1,70	1,70	1,47	1,46
110	1,94	1,94	1,74	1,74	1,51	1,50
120	1,98	1,98	1,77	1,77	1,54	1,54
130	2,01	2,01	1,80	1,80	1,57	1,57
140	2,04	2,04	1,83	1,83	1,60	1,60
150	2,07	2,07	1,86	1,86	1,62	1,62

Figure 4-11: Wind induced maximum pressure versus the height above ground level [21]

When the height of the building is known the maximum pressure on the building can be read from the table. This load generates a bending moment in the foot of the building per meter of the building.

$$M_w = C_w \left(\frac{1}{2} \cdot q_{w,foot} \cdot H^2 + \frac{1}{3} \left(q_{w.top} - q_{w,foot} \right) \cdot H^2 \right)$$

The corresponding static strength is now:

$$P_{st} = \frac{2}{H^2} \cdot \beta \cdot M_w \qquad \text{[kPa]}$$

Where:

M_w	= Bending moment	[kPa/m^2]
C_w	= Factor dependent on the angle of impact	[-]
q_w	= Wind pressure against the building	[kPa]
H	= Height of the building	[m]
β	= Safety factor	[-]

C_w depends on the friction and suction that can occur and is for horizontal working wind load maximum 1,8 [21]. For this study this value of 1,8 is used.

4.3.2 Natural frequency

The structure will begin to vibrate as a result of the load. The value of the natural frequency says something about the speed with which the structure responds to the load. Because of the very short loading time the structure will vibrate in its highest frequency. If the loading time would be long the frequency would be very low. Given the assumption that the building will not collapse instantly these high frequencies will damp out rather quick and only the lowest natural frequency will remain.

Mass-spring-system

The vibration or deformation of the structure can very simplified be schematised by a mass-spring system [7]. This is a very rough method. In a spring system the structure is simplified to a system of to points reduced masses connected by springs. The spring characteristic is determined by the stiffness of the structure. To formulate and solve the equation of the masses gives the spring forces and displacements. The loading time is very short, so the maximum response will be during the first vibration. Damping hardly plays a role during the first vibration. This method gives a very global picture of the damage therefore a detailed description of the structure is not necessary. The simplest spring system is a one-mass-spring system, which has one mass M and one spring with a spring stiffness K.

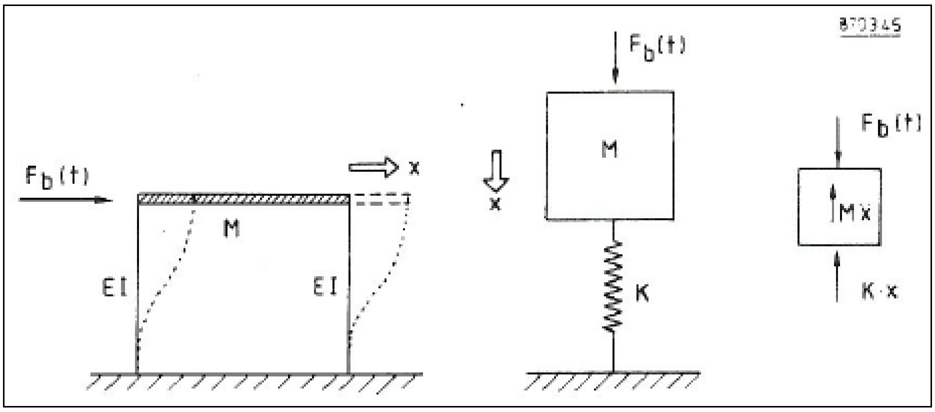


Figure 4-12: Examples of one-mass-spring systems

The equilibrium in a one-mass-spring system is described in the next equation:

$$M \cdot x'' + K \cdot x = F_b(t)$$

The frequency at which the structure will vibrate after an impact is its natural frequency f.

$$f = \frac{\omega}{2\pi}$$

$$\omega = \sqrt{\frac{K}{M}}$$

$$T = \frac{1}{f}$$

Where:

f	= Natural frequency	$[s^{-1}]$
ω	= Angular rotation	$[s^{-1}]$
K	= Spring stiffness	$[kg*s^{-2}]$
M	= Mass	[kg]
T	= Natural period	[s]

Several empirical methods to determine the natural period are available [7].

- A very rough method is:
  $$T = k_1 \cdot H$$
  The constant k_1 depends on the material of the building. For office buildings in the Netherlands k_1= 0,02.

- If also the depth of the building is accounted for the formula becomes:
  $$T = k_2 \cdot \frac{H}{\sqrt{L}}$$
  The value for k_2 = 0,09.

- For buildings with a framework structure:
  $$T = 0{,}1 \cdot n$$
  Where n is the number of floors.

- Yet another method for framework structures gives:
  $$T = k_3 \cdot H^{\frac{3}{4}}$$
  For steel k_3 = 0,085 and for concrete k_3 = 0,061.

Very high buildings have very low natural frequency. The Empire state building has a natural frequency of 0,12 Hz. One or two story wooden buildings have the highest natural frequency of 10 Hz

A more elaborate method to determine the natural period is on bases of its deformation.

$$T = \sqrt{\frac{0{,}25}{\delta}}$$

The method determines the natural frequency by finding the maximum deformation as a result of the de dead load of the building. The deformation of the building consist of two components; shear and bending.

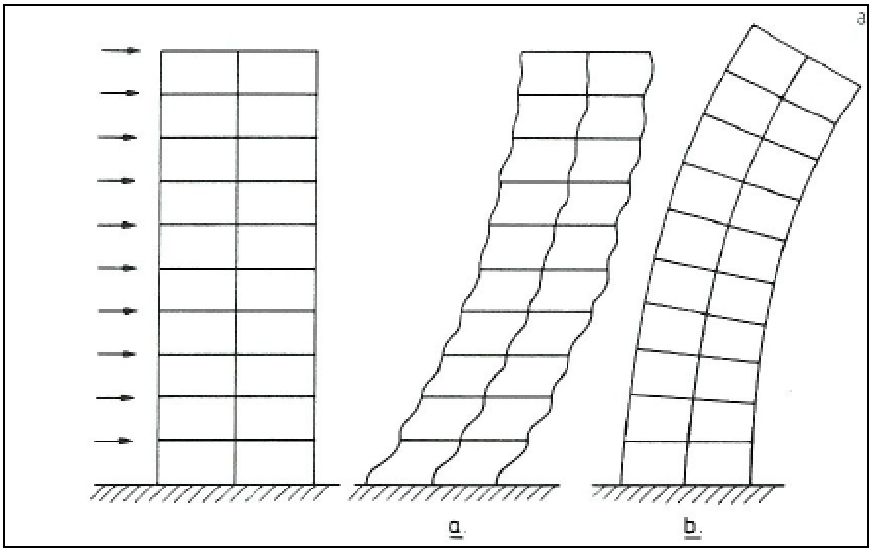


Figure 4-13: Deformation of a building by (a) shear and (b) bending

The floors are considered to be infinite stiff, which result in shear deformation. The whole structure including the stabilizing elements such as walls and cores act as a bending beam. The maximum deformation as a result of a horizontal working dead load has to be determined.

The deformation of a beam as a result of shear is:

$$\delta = \frac{p_{eg} \cdot n^2 \cdot h^2}{24 \cdot E \cdot \Sigma I_k}$$

The deformation of a beam as a result of bending is:

$$\delta = \frac{p_{eg} \cdot n^4 \cdot h^4}{4 \cdot E \cdot A_k \cdot L^2}$$

The dead load on one row of columns in the direction of the load is:

$$p_{eg} = \rho \cdot g \cdot B_k \cdot L$$

Where:

n	= Number of floors	[-]
h	= Column height	[m]
E	= Moment of elasticity	[Pa]
ΣI_k	= Summation of E of all the columns	[m^4]
A_k	= Cross section of one column	[m^2]
ρ	= Specific mass of building	[kg/m^3]
B_k	= Distance between columns	[m]
L	= Building depth	[m]

$$\Sigma I_k = n_k \cdot \frac{1}{12} \cdot b_k^4$$

4.3.3 Ductility

If a linear-elastic relation is assumed for the load and the force in the spring of a spring system then the displacement will be zero when the force is released; a linear-elastic force-displacement relation.

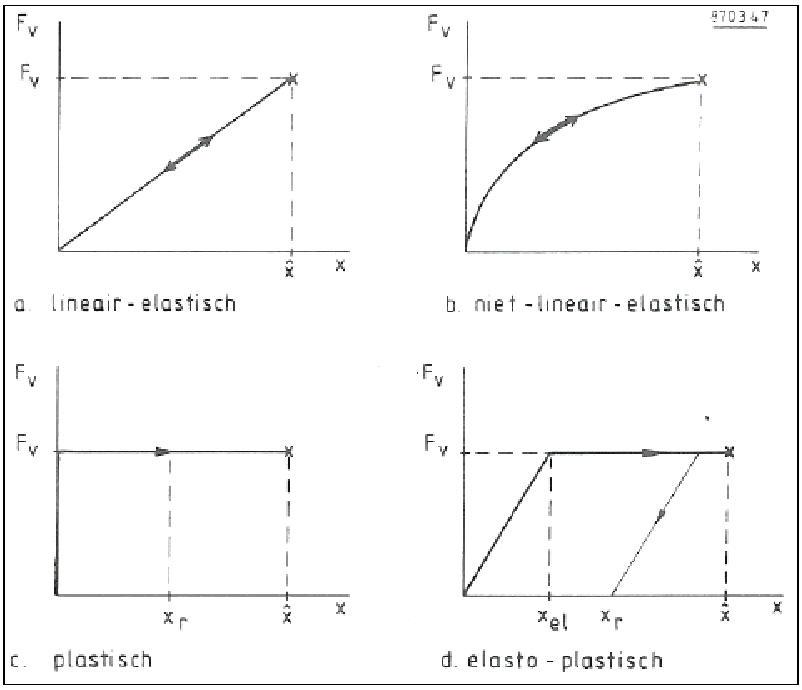


Figure 4-14: Force-displacement relations.

Most building materials show elastic-plastic behaviour. A building as a whole will also show this behaviour. The service load and wind load will be in the elastic zone. Higher loads, such as an explosion will be in the plastic zone or the building will collapse.

The building will show a plastic deformation. The structure will collapse if the maximum displacement is reached. A measure for the ability of a structure for plastic deformation is the ductility.

$$Du = \frac{\hat{x}}{\hat{x}_{el}}$$

$\hat{x} =$ maximum deformation

$\hat{x}_{el} =$ maximum elastic deformation

A large ductility means that the building is able of a large plastic deformation and/or a small elastic deformation.

4.3.4 Mechanical properties elements

This paragraph addresses the static strength, natural frequency and ductility for several structural elements. These elements are: a glass window, the concrete core, the floors and columns.

Static strength of elements

Several failure modes of a concrete element can be distinguished.

- First cracks in concrete.
- Yielding of reinforcement. The stresses in the element exceed the yielding strength of the reinforcement steel which results in plastic deformation.
- Fracture of reinforcement. The plastic deformation capacity is exceeded and the steel fractures.
- Failure of bond between concrete and steel.
- Exceeding of compression strength of concrete.
- Failure of the supports of the structure.

Not all of these failure modes result in the collapse of the structure. A concrete element will not fail when the first cracks appear. It is assumed that the building will collapse when the yield stress of the reinforcement is exceeded.

The focus of this study is on bending stresses. Short attention is paid to compression and shear stresses. Torsion or is not discussed.

For the calculations of the static strength use is made of [22], [23] and [24].

Static strength glass

According to [7] glass can crack already at an overpressure of 1 kPa, 50 % is broken at 3 kPa overpressure and all windows are shattered at an overpressure of 10 kPa. Car windows are shattered at 30 kPa. In contradistinction to a concrete element a window will fail after the first crack; no plastic deformation can occur. The static strength of glass calculated here will result in the total failure of the window.

Theoretical and experimental research resulted in a method to determine the theoretical fail strength of glass under a blast load [7]. The smallest discrepancy can initiate a crack. From theory and experiments it is known that the peak stress in glass can be found in the corners or in the middle of the window plane. How smaller the window how more overpressure it can withstand. The overpressure under which the glass fails can be presented by the following formulae.

$$q_{middle} = \frac{f_t}{6\pi^2 \cdot \alpha \cdot \left(\frac{a}{d}\right)^2 \cdot \left(1 + \nu\left(\frac{a}{b}\right)^2\right)}$$

$$q_{corner} = \frac{f_t}{6\pi^2 \cdot \alpha \cdot \left(\frac{a^3}{d^2 \cdot b}\right) \cdot (1 - \nu)}$$

Where:

f_t	= Theoretical failure strength	[Pa]
a	= Smallest dimension of window	[m]
b	= Largest dimension of window	[m]
d	= Thickness of window	[m]
v	= Poisson ratio	[-]

Factor α:

$$\alpha = \frac{16}{\pi^6 \cdot \left(1+\left(\frac{a}{b}\right)^2\right)^2}$$

The theoretical fail strength is

$$f_t = C_8 \cdot \left(\frac{d}{C_7}\right)^{-0,32} \cdot \left(\frac{b}{a}\right)^{0,47}$$

Where:
C_7 = 1 [m]
C_8 = 14,9 [MPa]

At a certain critical deformation the stresses in the middle of the glass will reduce because it will act as a membrane. When the deflection is larger than this critical deformation the stresses in the corners of the window will be larger and the corners will fail first. Otherwise the glass will fail in the middle. The critical deformation is:

$$\delta_{kr} = 6 \cdot \left(\frac{b}{a}\right)^{\frac{3}{2}} \cdot d$$

The deformation is:

$$\delta(q) = \frac{\alpha \cdot q \cdot a^4}{D}$$

Where D is the bend stiffness of the window:

$$D = \frac{E \cdot d^3}{12\left(1-v^2\right)}$$

Where E is the modulus of elasticity.

If the deformation in the middle of the window is larger than the critical deformation the glass will fail in the corners. The overpressure at failure is than:

$$\delta(q_{middle}) \geq \delta_{kr} : q = q_{corner}$$

If the deflection is smaller than the critical deflection the glass will fail in the middle. The overpressure at failure is than:

$$q = q_{middle} + \frac{\delta(q_{middle})}{\delta_{cr}} \cdot \left(q_{corner} - q_{middle}\right)$$

In case of double glazing the failure pressure will be larger by the following experimentally determined factor.

Factor = $\frac{d_1^3 + d_2^3}{d_1^3}$, with a maximum of 1,4.

Where,
d_1 = thickness thickest window [mm]
d_2 = thickness thinnest window [mm]

Static strength concrete core

In stead of determining the global static strength of a whole building the static strength of the concrete core of a building is determined here. The core is designed to withstand a horizontal wind load. The static strength as a result of this governing load is determined. After that the pressure on the façade is determined which results in the maximum tensile stresses being exceeded. Because the wind load is calculated with the safety factors in the ultimate limit state [21] this method also considers the plastic deformation of the core and therefore collapse of the building.

Wind load

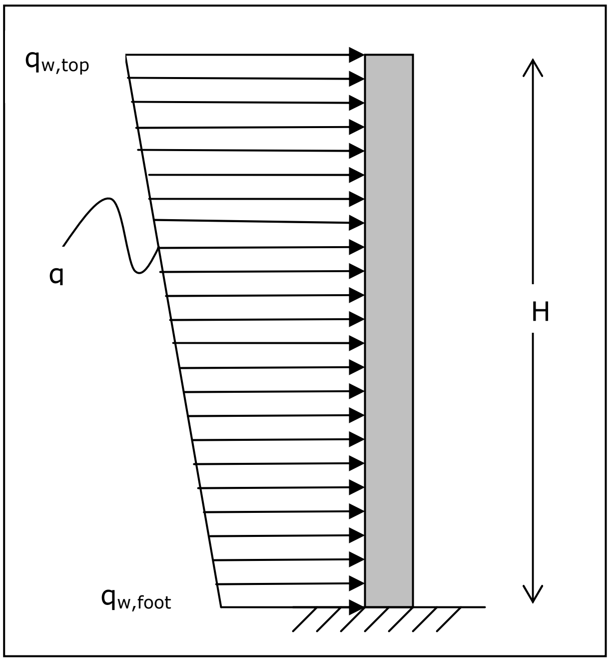


Figure 4-15: Schematization wind load on core

The governing load on the core is the wind load. This wind load results in a moment which is the same is for the global building.

$$M_w = C_w \left(\frac{1}{2} \cdot q_{w,foot} \cdot H^2 + \frac{1}{3} \left(q_{w.top} - q_{w,foot} \right) \cdot H^2 \right)$$

The corresponding static strength is now:

$$P_{st} = \frac{2}{H^2} \cdot \beta \cdot M_w$$

Where:
M_w = Bending moment [kPa/m^2]
C_w = Factor dependent on the angle of impact [-]
q_w = Wind pressure against the building [Pa]
H = Height of the building [m]
β = Safety factor [-]

C_w depends on the friction and suction that can occur and is maximum 1,8. For this study this value of 1,8 is used.

This method results in the same static strength as for the global building.

Stresses in the core

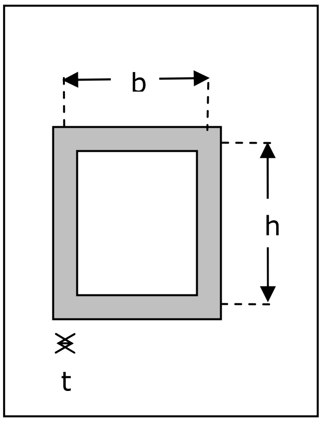


Figure 4-16: Schematization square concrete core

The mass moment of inertia of a rectangular concrete core is:

$$I_{yy} = \frac{1}{12} \cdot (b+t)^3 \cdot (h+t) - \frac{1}{12} \cdot (b-t)^3 \cdot (h-t)$$

Where:

b	= Width of core	[m]
h	= Height of core	[m]
t	= Thickness of wall	[m]

The compression stress in the core due to the weight of structure resting on the core is:

$$f_{e.g.} = \frac{N}{A}$$

The load of the building that is transferred to the core of the building is assumed to be:

$$N = b_{tr} \cdot l_{tr} \cdot h_{tr} \cdot G \cdot g$$

Where:

A	= Cross section of the core	[m^2]
b_{tr}	= Width of the section transferring the load on the core	[m]
h_{tr}	= Height of the section transferring the load on the core	[m]
l_{tr}	= Length of the section transferring the load on the core	[m]
G	= Average weight of the building	[kg/m^3]
g	= Gravitational acceleration	[m/s^2]

The concrete core will crack if the explosion load results in the maximum tensile stress in the core cross section being exceeded. This maximum tensile stress is f_b = 1,40 N/mm^2 for cracking of B35 concrete. The compression strength f_b' for B35 concrete is 21 N/mm2. These values can be multiplied by the safety factor β. The exceeding of the tensile strength will lead to cracks but not to collapse, the exceeding of the compression strength will lead to collapse. Because there is a compression stress in the core due to the dead weight of the building the maximum stress is:

Maximum tension stress: $f_{\max} = \beta \cdot \left(f_b + f_{e.g.} \right)$

Maximum compression stress: $f'_{\max} = \beta \cdot \left(f'_b - f'_{e.g.} \right)$

The stress in the cross section of the core can be determined with the following formula:

$$\sigma = \frac{1}{2} \cdot \frac{M \cdot b}{I_{yy}}$$

Where:

σ	= Stress		[N/m^2]
M	= Moment		[Nm]
b	= Width of core		[m]
I_{yy}	= Mass moment of inertia in direction of load		[m^4]

The static strength is the evenly distributed load on the building

$$P_{st} = \frac{q}{L} = \frac{2 \cdot M}{L \cdot H^2} = \frac{2 \cdot \sigma \cdot 2 \cdot I_{yy}}{H^2 \cdot b \cdot L} = \frac{4 \cdot \sigma \cdot I_{yy}}{H^2 \cdot b \cdot L}$$

$$P_{st,core,cracking} = \frac{4 \cdot \beta \cdot \left(f_b + f_{e.g.} \right) \cdot I_{yy}}{H^2 \cdot b \cdot L}$$

$$P_{st,core,compression} = \frac{4 \cdot \beta \cdot \left(f'_b - f'_{e.g.} \right) \cdot I_{yy}}{H^2 \cdot b \cdot L}$$

Where

P_{st}	= Static strength
q	= Evenly distributed load over the height of the building
L	= Length of the building
H	= Height of the building

Note 1:
This method describes the static strength as an evenly distributed load over the height of the building. In reality the load is not evenly distributed. On the foot of the building the load is larger than in the top.

Stresses in reinforcement steel

Not all the applied bending reinforcement in the core will be activated when the horizontal blast load acts on the building. It is assumed that only 1/3 of the reinforcement is activated.

The maximum moment in the steel reinforcement of the core is

$$M = \frac{1}{3} \cdot A_s \cdot f_s \cdot \beta \cdot b = \frac{1}{3} \cdot \omega_0 \cdot A_{core} \cdot f_s \cdot \beta \cdot b$$

Where:

M	= Moment of in the foot of the building	[Nm]
A_s	= Cross section of steel reinforcement	[m^2]
ω_0	= Percentage of reinforcement	[%]

A_{core} = Cross section concrete core [m^2]
f_s = Tensile strength [N/mm^2]
β = Safety factor [-]
b = Width of the core [m]

The moment in the foot of the core is:

$$M = \frac{1}{2} \cdot q \cdot H^2 = \frac{1}{2} \cdot P_{st} \cdot L \cdot H^2$$

Where:
L = Length of the building [m]
H = Height of the building [m]

$$P_{st,core,yielding} = \frac{2 \cdot M}{L \cdot H^2} = \frac{2 \cdot \frac{1}{3} \cdot \omega_0 \cdot A_{core} \cdot f_s \cdot \beta \cdot b}{L \cdot H^2} = \frac{8 \cdot \omega_0 \cdot b^2 \cdot t \cdot f_s \cdot \beta}{3 \cdot L \cdot H^2}$$

When this static strength is exceeded the reinforcement in the core will start to yield. The building can than be considered collapsed.

Shear stresses

When using the wind load method the shear force in the core is:

$$V = C_w \cdot H \cdot B \cdot \left(q_{w,foot} + \frac{1}{2} \cdot \left(q_{w,top} - q_{w,foot} \right) \right)$$

$$P_{st} = \frac{V}{B \cdot H} \cdot \beta = C_w \cdot \beta \cdot \left(q_{w,foot} + \frac{1}{2} \cdot \left(q_{w,top} - q_{w,foot} \right) \right)$$

Where:
V = Shear force [N]
C_w = Factor dependent on the angle of impact [-]
H = Height of the building [m]
B = Width of the building [m]
$q_{w,foot}$ = Wind pressure at the foot of the building [Pa]
$q_{w,top}$ = Wind pressure at the top of the building [Pa]
β = Safety factor [-]

The stresses in the cross section of the core can be calculated with the following formula.

$$\sigma = \frac{V_y \cdot S_y}{b \cdot I_{yy}}$$

Where:
V_y = Shear force [N]
S_y = Static moment [m^3]
b = Width shear cross section [m^4]

$$V = \frac{\sigma \cdot 2 \cdot t \cdot I_{yy}}{S}$$

$$S_y = (h - t) \cdot t \cdot \tfrac{1}{2} \cdot b$$

$$I_{yy} = \frac{1}{12} \cdot (b+t)^3 \cdot (h+t) - \frac{1}{12} \cdot (b-t)^3 \cdot (h-t)$$

The maximum shear stress in a cross section with shear reinforcement for B35 is 4,2 N/mm^2. Because this is the maximum shear stress *with* reinforcement the core can be considered collapsed when this value is exceeded.

$$P_{st} = \frac{V}{B \cdot H} \cdot \beta = \frac{\beta \cdot \sigma \cdot 2 \cdot t \cdot I_{yy}}{B \cdot H \cdot S} = \frac{\beta \cdot \tau \cdot 2 \cdot t \cdot \left(\frac{1}{12} \cdot (b+t)^3 \cdot (h+t) - \frac{1}{12} \cdot (b-t)^3 \cdot (h-t)\right)}{B \cdot H \cdot (h-t) \cdot t \cdot \frac{1}{2} \cdot b}$$
$$= \frac{\beta \cdot \tau \cdot \left((b+t)^3 \cdot (h+t) - (b-t)^3 \cdot (h-t)\right)}{B \cdot H \cdot (h-t) \cdot 3 \cdot b}$$

In case b is equal to h this becomes:

$$P_{st} = \frac{\beta \cdot \tau \cdot \left((b+t)^4 - (b-t)^4\right)}{B \cdot H \cdot (b-t) \cdot 3 \cdot b}$$

The maximum shear stress in the other direction is:

$$S_y = t \cdot b \cdot \tfrac{1}{2} \cdot b + t \cdot \tfrac{1}{2} \cdot b \cdot \tfrac{1}{4} \cdot b = \tfrac{5}{8} \cdot t \cdot b^2$$

$$P_{st} = \frac{V}{B \cdot H} \cdot \beta = \frac{\beta \cdot \sigma \cdot 2 \cdot t \cdot I_{yy}}{B \cdot H \cdot S} = \frac{\beta \cdot \tau \cdot 2 \cdot t \cdot \left(\frac{1}{12} \cdot (b+t)^4 - \frac{1}{12} \cdot (b-t)^4\right)}{B \cdot H \cdot \frac{5}{8} \cdot t \cdot b}$$
$$= \frac{4 \cdot \beta \cdot \tau \cdot \left((b+t)^4 - \cdot (b-t)^4\right)}{15 \cdot B \cdot H \cdot b} = \frac{4 \cdot 2{,}7 \cdot 4{,}2 \cdot \left((b+t)^4 - \cdot (b-t)^4\right)}{15 \cdot B \cdot H \cdot b}$$

Static strength floors

The floor slab can be considered to be a high beam. Usually the floors are equipped with tensile strands to be able to transfer the forces to the walls and cores of the building.

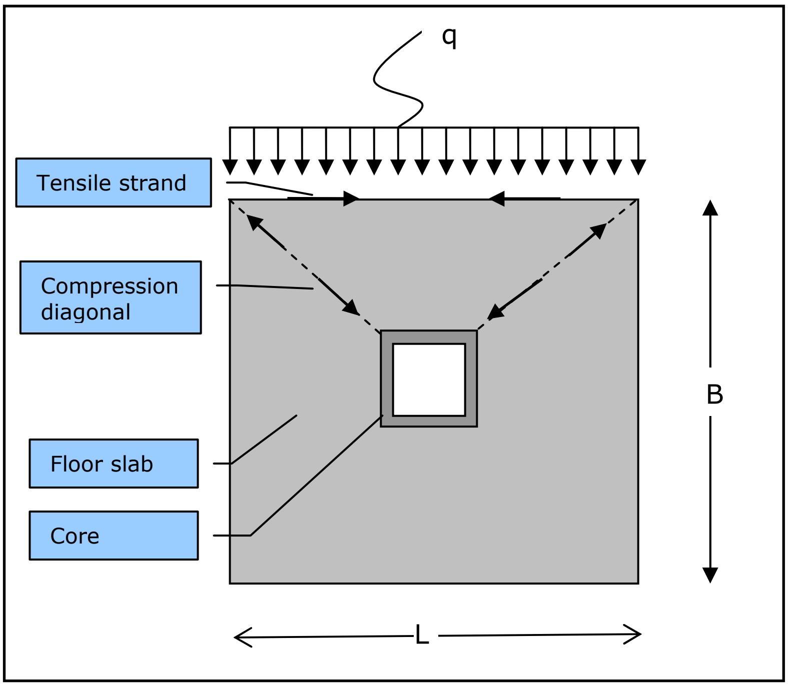


Figure 4-17: Load on the floor

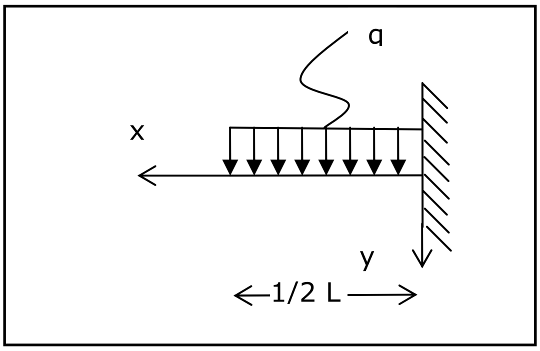


Figure 4-18: Schematization load on the floor

If the floor has to transfer the load to a central core the maximum moment in the floor slab is:

$$M = \frac{1}{2} \cdot L \cdot q \cdot \frac{1}{4} \cdot L = \frac{1}{8} \cdot q \cdot L^2$$

Where:
q = Evenly distributed load over the length of building [N/m]
L = Length of the building [m]

The evenly distributed load q as a result of the explosion load is:

$$q = P_{st} \cdot S$$

Where:
S =Story height [m]
P_{st} =Static strength of floor [N/m^2]

Wind load

Just as the strength of the global building and the core of the building the static strength of the floors can be calculated using the governing wind load.

The wind pressure q_w depends on the height of the floor in the building. See table in paragraph 4.3.1. The static strength based on the governing wind load is now:

$$P_{st,wind} = C_w \cdot \beta \cdot \frac{q}{S} = C_w \cdot \beta \cdot \frac{8 \cdot M}{S \cdot L^2} = C_w \cdot \beta \cdot \frac{8}{S \cdot L^2} \cdot \frac{q_w \cdot S \cdot L^2}{8} = C_w \cdot \beta \cdot q_w$$

Stresses in floor

In stead of using the governing wind load the static strength can also be determined by calculating the maximum stresses in its cross section.

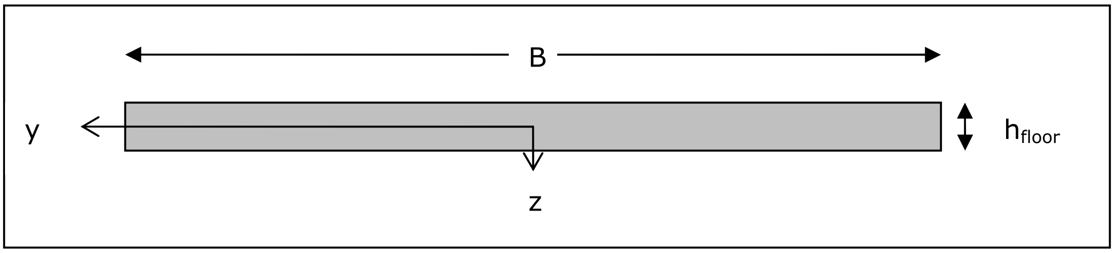


Figure 4-19: Cross section concrete floor

The mass moment of inertia is:

$$I_{yy} = \frac{1}{12} \cdot h_{floor} \cdot B^3$$

Where:
h_{floor} = Height of floor [m]
B = Width of the building [m]

The stress in the floor slab due to bending can be determined with the following formula:

$$\sigma = \frac{1}{2} \cdot \frac{M \cdot B}{I_{yy}}$$

Where:
σ = Stress [N/m^2]
M = Moment [Nm]
B = Width of building [m]
I_{yy} = Mass moment of inertia in direction of load [m^4]

The maximum tensile stress in concrete is 1,4 N/mm^2. This value can be multiplied by the safety factor β.

$$f_{tot} = f_b \cdot \beta$$

When these formulas are combined this results for the static strength in:

$$P_{st} = \frac{q}{S} = \frac{8 \cdot M}{S \cdot L^2} = \frac{8}{S \cdot L^2} \cdot \frac{2 \cdot \sigma \cdot I_{yy}}{B} = \frac{16 \cdot f_{tot} \cdot \frac{1}{12} \cdot h_{floor} \cdot B^3}{S \cdot L^2 \cdot B} = \frac{4 \cdot f_b \cdot \beta \cdot h_{floor} \cdot B^2}{3 \cdot S \cdot L^2}$$

This static strength represents the point at which the concrete floor will start cracking.

Stresses in reinforcement steel

The floor is loaded in the y-direction and therefore not all the applied bending reinforcement will be activated when the horizontal blast load acts on the building. It is assumed that only 1/3 of the reinforcement is activated.

The maximum moment in the steel reinforcement of the floor is

$$M = \frac{1}{3} \cdot A_s \cdot f_s \cdot \beta \cdot 0{,}9 \cdot B$$

Where:

A_s	= Cross section of steel reinforcement	[m^2[
f_s	= Tensile strength	[N/mm^2]
β	= Safety factor steel	[-]
B	= Width of the building	[m]

$$P_{st} = \frac{q}{S} = \frac{8 \cdot M}{S \cdot L^2} = \frac{8 \cdot A_s \cdot f_s \cdot \beta \cdot 0{,}9 \cdot B}{3 \cdot S \cdot L^2} = \frac{8 \cdot h_{floor} \cdot B^2 \cdot \omega_0 \cdot f_s \cdot \beta \cdot 0{,}9}{3 \cdot S \cdot L^2}$$

When this static strength is exceeded the reinforcement in the floor will start to yield. The building can than be considered collapsed.

Note 1:
Only bending is discussed here. There is no attention for normal forces, shear or torsion.

Note 2:
This method describes the static strength as an evenly distributed load over the length of the floor. In reality the load is not evenly distributed.

Shear stresses in floor

When using the wind load method the shear force in the floor is:

$$V = \frac{1}{2} \cdot L \cdot S \cdot C_w \cdot q_w$$

$$V = \frac{1}{2} \cdot L \cdot S \cdot P_{st}$$

$$P_{st,floor,wind,shear} = \frac{2 \cdot V}{L \cdot S} \cdot \beta = \frac{2 \cdot L \cdot S \cdot C_w \cdot q_w}{2 \cdot L \cdot S} \cdot \beta = C_w \cdot q_w \cdot \beta$$

Where:

V	= Shear force	[N]
C_w	= Factor dependent on the angle of impact	[-]
q_w	= Wind pressure at the building	[Pa]
S	= Story height of the building	[m]
β	= Safety factor	[-]

The maximum shear stress in a rectangular cross sections is:

$$\tau_{\max} = \frac{3 \cdot V}{2 \cdot b \cdot h}$$

$$V = \frac{1}{2} \cdot L \cdot S \cdot P_{st}$$

$$P_{st,floor,stress,shear} = \frac{2 \cdot V}{L \cdot S} \cdot \beta = \frac{2 \cdot \tau_{\max} \cdot 2 \cdot h_{floor} \cdot B \cdot \beta}{3 \cdot L \cdot S}$$

The maximum shear stress in a cross section with shear reinforcement for B35 is 4,2 N/mm^2. Because this is the maximum shear stress *with* reinforcement the floor can be considered collapsed when this value is exceeded.

Static strength of columns

When the glass fails the blast wave enters the building, where it acts as load on the columns and walls inside the building.

Stresses in column

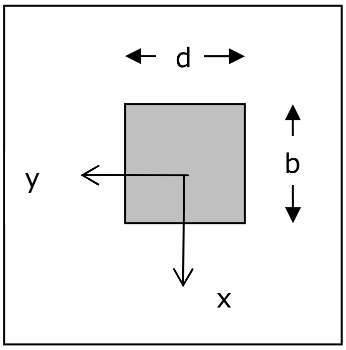


Figure 4-20: Schematization of column

The mass moment of inertia for a square column is:

$$I_{yy} = \frac{1}{12} \cdot b \cdot d^3$$

Where:

b	= Width of the column	[m]
d	= Depth of the column	[m]

In the columns and walls of the building there is besides the horizontal working load of the explosion also the dead load of the building. But these elements are only loaded when the blast wave has entered the building. The overpressure will push the floors down and the ceiling up. It is assumed that this cancels out the normal force in the columns and walls.

The stresses in the column are:

$$\sigma = \frac{M \cdot d}{2 \cdot I_{yy}}$$

Where:

M	= Maximum moment	[Nm]
b	= Width of the column	[m]
I_{yy}	= Mass moment of inertia in the direction of the load	[m^4]

Because it is assumed that the overpressure results in an evenly distributed load the maximum moment is:

$$M = \frac{1}{8} \cdot q \cdot l^2$$

Where:

q	= Evenly distributed load over the length	[N/m]
l	= Length of the column or wall	[m]

The evenly distributed load q as a result of the explosion load is:

$$q = P_{st} \cdot b$$

Where:

P_{st}	= Static strength of beam	[kPa]
b	= Width of the column or wall	[m]

The maximum tensile stress in concrete is 1,4 N/mm2. This value can be multiplied by the safety factor β. When this value is exceeded the concrete column will start to crack.

$$f_{tot} = f_b \cdot \beta$$

When these formulas are combined this results for the static strength in:

$$P_{st,column,cracking} = \frac{q}{b} = \frac{8 \cdot M}{b \cdot l^2} = \frac{8}{b \cdot l^2} \cdot \frac{2 \cdot \sigma \cdot I_{yy}}{d} = \frac{16 \cdot f_{tot} \cdot \frac{1}{12} \cdot b \cdot d^3}{l^2 \cdot b \cdot d} = \frac{4 \cdot f_b^{'} \cdot \beta \cdot d^2}{3 \cdot l^2}$$

Stresses in reinforcement steel of column

The maximum moment in the steel reinforcement of the column or wall is

$$M = \frac{1}{3} \cdot A_s \cdot f_s \cdot \beta \cdot 0{,}9 \cdot b = \frac{1}{3} \cdot \omega_0 \cdot b \cdot f_s \cdot \beta \cdot 0{,}9 \cdot d^2$$

Where:

M	= Moment in the column	[Nm]
A_s	= Cross section of steel reinforcement	[m^2[
ω_0	= Percentage of reinforcement	[%]
A	= Cross section	[m]
f_s	= Tensile strength	[N/mm^2]
β	= Safety factor	[-]
b	= Width of the element	[m]
d	= Depth of the element	[m]

The maximum moment is

$$M = \frac{1}{8} \cdot q \cdot l^2 = \frac{1}{2} \cdot P_{st} \cdot b \cdot l^2$$

Where:

l	= Length of the element	[m]
b	= Width of the element	[m]

$$P_{st,column,yielding} = \frac{2 \cdot M}{b \cdot l^2} = \frac{2 \cdot \frac{1}{3} \cdot \omega_0 \cdot b \cdot f_s \cdot \beta \cdot 0{,}9 \cdot d^2}{b \cdot l^2} = \frac{2 \cdot \omega_0 \cdot f_s \cdot \beta \cdot 0{,}9 \cdot d^2}{3 \cdot l^2}$$

When this static strength is exceeded the reinforcement in the column will start to yield. The building can than be considered collapsed.

Shear stresses in column

The maximum shear stress in a rectangular cross sections is:

$$\tau_{\max} = \frac{3 \cdot V}{2 \cdot b \cdot d}$$

$$V = \frac{1}{2} \cdot l \cdot b \cdot P_{st}$$

$$P_{st,column,stress,shear} = \frac{2 \cdot V}{l \cdot b} \cdot \beta = \frac{2 \cdot \tau_{\max} \cdot 2 \cdot b \cdot d \cdot \beta}{3 \cdot l \cdot b} = \frac{4 \cdot \tau_{\max} \cdot d \cdot \beta}{3 \cdot l}$$

The maximum shear stress in a cross section with shear reinforcement for B35 is 4,2 N/mm^2. Because this is the maximum shear stress *with* reinforcement the column can be considered collapsed when this value is exceeded.

Columns in facade

When the columns are placed in the façade and the blast load on the façade elements transfers directly to the columns the static strength of the column will be smaller. In stead of a blast working in the width of the column it works on the façade element over a width as large as the distance between two columns, which in general is the beam length. The factor with which the static strength of a column inside the building can be multiple to get the static strength of a column when it is placed in the façade is the following.

$$c_{facade} = \frac{b_{column}}{l_{beam}}$$

Natural frequency of elements

Natural frequency of glass

The lowest Eigen frequency for a plate is:

$$f = \frac{\pi}{2}\left(\frac{1}{a^2} + \frac{1}{b^2}\right) \cdot \sqrt{\frac{E \cdot I}{\rho \cdot d \cdot (1 - \nu^2)}}$$

Where:

E	= Modulus of elasticity	[Pa]
I	= Moment of inertia	[m^4]
a	= Glass plate length	[m]

b = Glass plate width [m]
d = Glass plate thickness [m]
v = Poisson's ratio [-]

Natural frequency of core

The core of the building will always respond together with the rest of the building. The natural frequency of the core is therefore considered to be identical to the natural frequency of the whole building.

Natural frequency of floors

The natural frequency of plates is given by the following formula [7]

$$f = \frac{\pi}{2} \cdot \left(\frac{1}{a^2} + \frac{1}{b^2} \right) \cdot \sqrt{\frac{E \cdot I}{\rho \cdot d \cdot \left(1 - v^2\right)}}$$

Where:
a = Length of the plate [m]
b = Width of the plate [m]
E = Modulus of elasticity [N/m^2]
I = Moment of inertia [m^4]
ρ = Dead weight of the plate [kg/m^3]
d = Thickness of the plate [m]
v = Poisson's ratio [-]

$$I_{yy} = \frac{1}{12} \cdot a \cdot d^3$$

Natural frequency columns

If the material properties are known the lowest natural frequency for a column is:

$$f = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{E \cdot I}{\rho \cdot A_l}}$$

Where:
E = Modulus of elasticity [Pa]
I = Moment of inertia [m^4]
l = Column length [m]
A_l = Column cross section [m^2]

<u>Ductility of elements</u>

Ductility glass

Glass is very brittle and therefore has a ductility of 1.

Ductility of concrete core

The core of the building will always respond together with the rest of the building. The ductility of the core is therefore considered to be identical to the ductility of the whole building.

Ductility of floors

There is no method known to determine or estimate the ductility of floors. In this study it is assumed that the ductility of floors is the same as that of the whole building or the core; 4 to 10.

Ductility of columns

Empirical determined values for the ductility of beams is presented in this table.

Structure	Ductility
Reinforced concrete beams	$\frac{0,1}{\omega_0 - \omega_0'}$
Steel beams, loaded on bending	26,4
Steel beams, loaded on bending and compression	8,1
Welded portals, vertically loaded	6-16
Composed T-beam	8

Table 4-2: Ductility for several elements [7]

The value ω_0 is the percentage of tensile reinforcement steel and ω'_0 is the percentage of compression reinforcement steel.

The ductility of the columns is considered to be comparable to the ductility of the beams.

4.4 Damage

When the load on the building and mechanical properties of the structure are known a prediction can be made of the damage. To give a reference of how much damage can occur the damage levels and the amount of damage is assessed in paragraph 4.4.1. and 4.4.2.. After that calculation methods are given to determine the damage. Also the damage to separate elements is assessed. Finally something will be said about progressive collapse.

4.4.1 Damage levels

On basis of empirical data three damage levels have been set up in [7].

- Serious / severe damage
  There is so much damage that the structure cannot be used any more unless the building gets a complete restoration. With severe damage is meant that several load bearing structures and part of the building have collapsed. Walls that have not collapsed are heavily damaged and cracked. In general the whole structure will have to be demolished. (ultimate limit state)

- Moderate damage
  Important components of the structure are damaged in such a way that normal use of the building is not possible unless large reparations are carried out. With moderate damage the structure still can be used, but the walls will be cracked and unreliable and the load bearing structure is damaged and dislocated. (service limit state)

- Light damage
  Windows are broken, walls and roofs are have light damage. The furniture is blow away. Little repair is enough the make the building fit for normal use again.

4.4.2 Amount of damage

In this paragraph a relation is given between the amount of damage and the peak overpressure on bases of existing literature. The values presented here are not used in this study and are only as reference for the answers.

Damage level	P_s (kPa)
Total destruction	> 83
Severe damage	> 35
Moderate damage	> 17
Light damage	> 3,5

Table 4-3: Building damage levels for peak overpressure P_s [7]

Damage description	P_s (kPa)
Failure connection steel or aluminium corrugated plates	7-14
Wall of concrete blocks collapsed	15-20
20-30 cm stone wall collapsed	50
Light damage to steel framework	8-10
Collapse of steel framework and displacement of foundation	20
Steel office building without framework collapsed	20-30
Sheeting of light office building ripped away	30
Roof of a storage tank collapsed	7
Support of a round storage tank collapsed	100
Rupture of an empty oil storage tank	20-30
Displacement of round storage tank, connected pipes collapsed	50-100
Damage to fractionation column	35-80
Light deformation to pipe bridge	20-30
Displacement of pipe bridge, breaking off pipes	35-40
Collapse of pipe bridge	40-55
Bodywork of cars trucks smashed	35
Wooden telephone poles broken	35
Loaded railway carriages knocked over	50
Large trees fallen over	20-40

Table 4-4: Overpressure and damage on structures from different literature [7]

The values from this table are based on heavy artillery from WWII and therefore these values should be used with caution when comparing them with a BLEVE. No values for the impulse are given here which also reduces its usefulness.

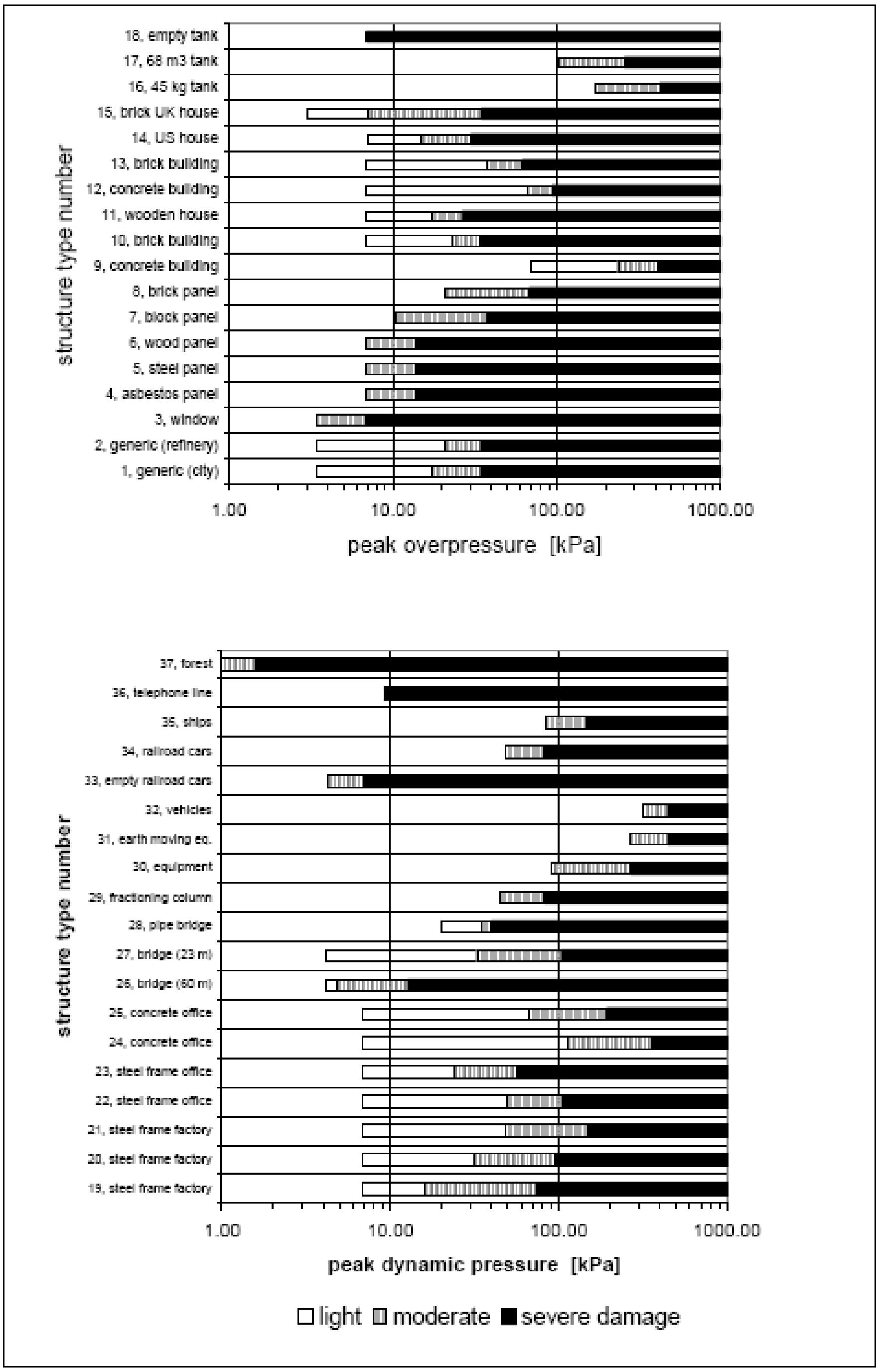


Figure 4-21: Damage of structures [7]

Damage description	P_s (kPa)
Glass crack	1
Some damage on ceilings	2
Limited minor structural damage	3
Considerable damage to houses	7
Partial collapse of roofs and walls of houses	15
Cracking of storage tank	20-30
Collapse of steel frame buildings	50
Total collapse of all buildings	70

Table 4-5: Damage according to Baker [7]

4.4.3 Chance of collapse

With probit functions the chance of damage can be determined analytically. In [7] this method is described. For buildings higher than four floors the probit on (partial) collapse as a result of a shock wave is give by the following formulae.

$$\Pr = 5 - 2{,}92 \cdot \ln V$$

$$V = \left(\frac{0{,}9}{\bar{P}}\right)^{1{,}4} + \left(\frac{3}{\bar{i}}\right)^{2{,}7}$$

The chance of collapse can be read out from the table below.

%	0	1	2	3	4	5	6	7	8	9
0	--	2.67	2.95	3.12	3.25	3.36	3.45	3.52	3.59	3.66
10	3.72	3.77	3.82	3.897	3.92	3.96	4.01	4.05	4.08	4.12
20	4.16	4.19	4.23	4.26	4.29	4.33	4.36	4.39	4.42	4.45
30	4.48	4.50	4.53	4.56	4.59	4.61	4.64	4.67	4.69	4.72
40	4.75	4.77	4.80	4.82	4.85	4.87	4.90	4.92	4.95	4.97
50	5.00	5.03	5.05	5.08	5.10	5.13	5.15	5.18	5.20	5.23
60	5.25	5.28	5.31	5.33	5.36	5.39	5.41	5.44	5.47	5.50.
70	5.52	5.55	5.58	5.61	5.64	5.67	5.71	5.74	5.77	5.81
80	5.84	5.88	5.92	5.95	5.99	6.04	6.08	6.13	6.18	6.23
90	6.28	6.34	6.41	6.48	6.55	6.64	6.75	6.88	7.05	7.33
--	0.0	0.1	0.3	0.3	0.4	0.5	0.6	0.7	0.8	0.9
99	7.33	7.37	7.41	7.46	7.51	7.58	7.65	7.75	7.88	8.09

Table 4-6: Chance of collapse for probit

The scaled impulse and scaled pressure must be subsidized.

$$\bar{i} = \frac{I \cdot \omega}{P_{st}}$$

$$\bar{P} = \frac{P}{P_{st}}$$

Where:

I	= Impulse	[kPa*s]
ω	= Angular rotation	[s^{-1}]
P_{st}	= Static strength	[kPa]
P	= Reflected peak overpressure or side on peak overpressure	[kPa]

4.4.4 Equivalent static load

In stead of determining the structural response with methods for dynamic loading it is also possible to use a method for static loads with addition of the Dynamic Load Factor, DLF. With this equivalent-static calculation method the maximum values for displacements and forces can be calculated. The DLF can be determined with a spring system with linear elastic deformation.

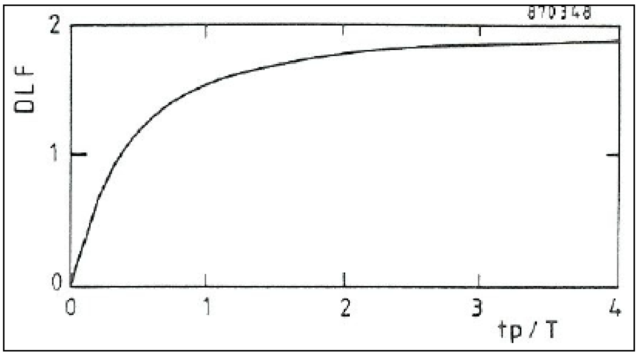


Figure 4-22: DLF for a linear-elastic structure under load of a shock wave

If tp << T than one can speak of an impulse load in stead of a step load when tp >> T, where the loading is so slow that it is a static load. The maximum dynamic deflection is obtained by solving the differential equation for a linear elastic one-mass-spring system [7].

$$\hat{x} = \frac{I}{m \cdot \omega}$$

Where:

I	= Impulse	[kPa*s]
m	= Building mass	[kg/m^2]
ω	= Angular rotation	[s^{-1}]

The static deflection is:

$$\hat{x}_{st} = \frac{P}{K} = \frac{P_r}{m \cdot \omega^2}$$

$$DLF = \frac{\hat{x}}{\hat{x}_{st}} = \frac{I / m \cdot \omega}{P_r / m \cdot \omega^2} = \frac{I \cdot \omega}{P_r}$$

$$P_{eq.staticload} = DLF \cdot P_r = I \cdot \omega$$

If the equivalent static load is larger than the static strength the building will collapse.

4.4.5 Required ductility

The ductility that the building needs to take on the imposed impulse can be determined.

$$i = \frac{P_{st}}{\omega}\sqrt{2 \cdot Du - 1}$$

$$Du_{required} = \frac{1}{2}\left(\left(\frac{i \cdot \omega}{P_{st}}\right)^2 + 1\right)$$

The building will collapse if the required Du is larger than the Du of the building.

4.4.6 Damage to elements

There are three methods to asses the damage of the building elements: Chance of collapse, equivalent static load and required ductility [7].

Chance of collapse

The probit function is only for buildings with more than four floors. It cannot be applied to separate elements. Except for glass for which a separate equation is given.

$$\mathrm{Pr} = -16{,}58 + 2{,}53 \cdot \ln Ps$$

According to the probit function there is a 1 % chance of failure with an overpressure of 2 kPa and a 50 % change with 5 kPa.

Equivalent static load

The equivalent static load is the same for the global building as for the different elements. It only changes because the impulse and pressure differ from location to location on the building, because the distance increases.

For the glass, core, beam and column the static strength is determined with different methods; wind load, stresses in concrete and stresses in reinforcement steel. There is no damage if the calculated static strength is larger than the equivalent static load.

$$P_{st} \geq P_{eq.staticload}$$

Ductility

The required ductility can be determined for the elements in the same way as for the global building. Furthermore, when the positive phase duration, the natural period and the DLF are known, the required ductility can be determined using the following diagrams.

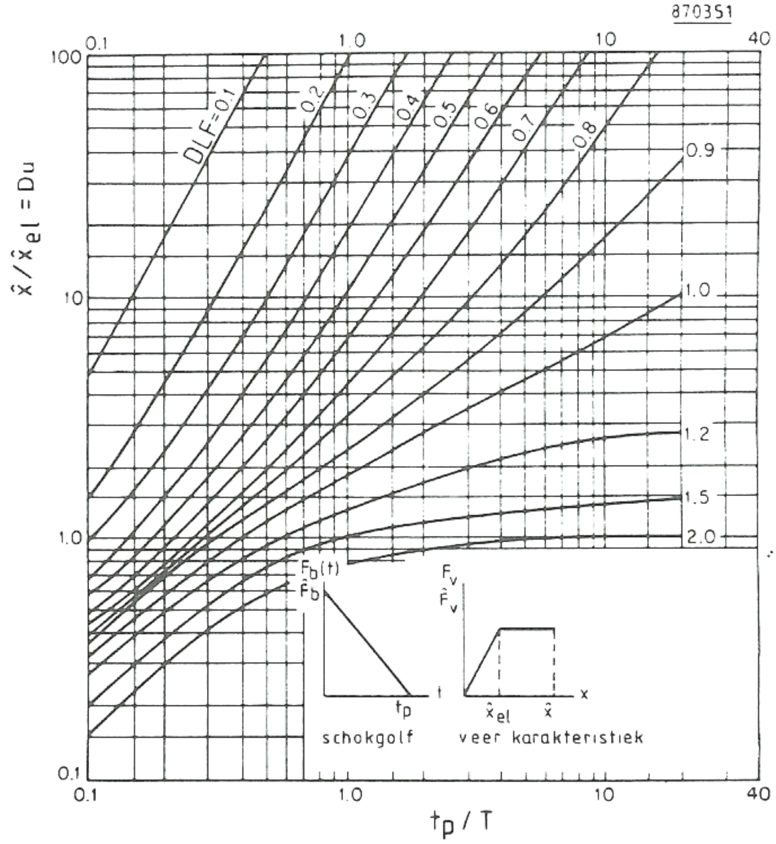


Figure 4-23: Response of an elastic-plastic one-mass-spring system to a shock wave [7].

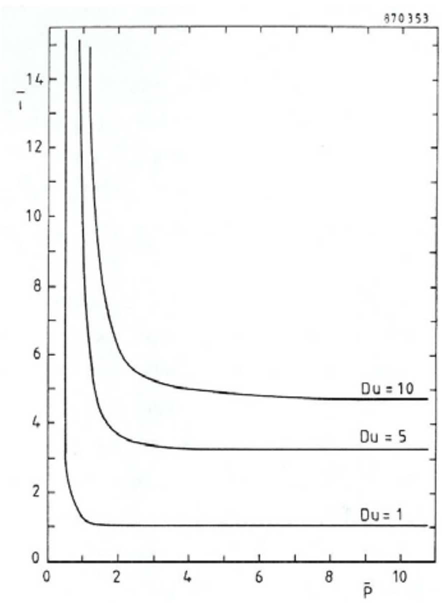


Figure 4-24: Scaled pressure impulse diagram

For the use of these diagrams the DLF (Dynamic Load Factor) is used. This factor is explained in paragraph 4.4.4. Because this diagram method is based on a one-mass-spring system the question rises if it is applicable on a whole structure. Research [20] has shown that it is applicable if $t_p/T > 0{,}1$ and if the load is distributed evenly over the surface of the building.

In total there are three methods to determine the required ductility: the formula also used for the global building and the two diagrams given here. These can be applied to every element. If the ductility of the core is lower the required ductility the structure will fail.

$$Du \geq Du_{required}$$

4.4.7 Progressive collapse

A building should have some sort of structural redundancy. If one column fails for whatever reason this shouldn't result in the collapse of the building or disproportionate damage. The loads that were carried by a collapsed column should be redirected to the columns beside it forming a second load path. When designing the beams and columns of the building this second load path should be accounted for.

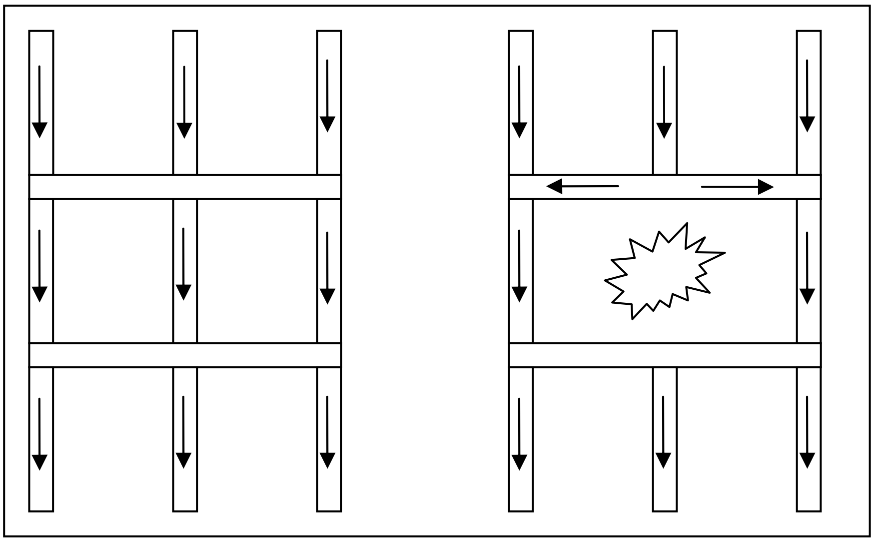


Figure 4-25: Second load path

The progressive collapse of every building is different and therefore despite a preferred second load path the failure of only one beam or column could initiate a progressive collapse. Also if the whole front row of columns collapses this could result in the total collapse of the building. Because the explosion of an LPG tank truck always occurs on ground level the maximum forces are on ground level. It's logic that multiple columns and beams in the same area of the building will fail. The damage to a building on lower floors is expected to be higher than on higher floors. All the dead load of the upper floors has to be transferred downward while multiple columns and beams are missing. The worst case scenario is that the explosion destroys a part of building but that because the integrity of the building structure is damaged it will collapse entirely.

With the methods used in this study it is not possible to determine the load on beams and columns beyond the facade of the building. The stresses and thus the damage on the second row of columns and beams are unknown. If the methods to determine the damage to the elements indicate that all the beams and columns in the façade will fail, the whole façade of the building will collapse. It is save to say that in that case also at least a part of the columns and beams behind the façade will collapse. If the used methods indicate that the whole building will fail before the elements will fail than there is no progressive collapse; the whole building will collapse.

So the problem is that it is not known how many elements *inside* the building will collapse and how many lost elements initiate progressive collapse or even total collapse. Further study is recommended. In essence what you want is a more elaborate damage categorization. The relation between the amount of lost elements and the effect on the building structure integrity should be assessed.

A solution could be that buildings are designed to allow for the collapse of a complete façade or row of columns. If these fail the rest of building keeps standing. For this reason the stabilizing elements or other important structural elements should not be near the façade of the building.

4.5 Method evaluation

4.5.1 Sensitivity analysis

The response of a building to the explosion is determined by the blast load, response aspects, mechanical properties and damage, which are dealt with in the previous chapters, respectively 3, 4.2, 4.3 and 4.4. A sensitivity analysis of the methods used to determine these aspects and properties results in an idea which parameters have a large influence on the response of the building.

What is the sensitivity of the parameters? When you change a parameter, what does that mean for the outcome of the methods? When a certain parameter has a very low influence on the outcome of a method it is not necessary to know the exact value for that parameter. What parameters have a large influence on the blast resistance of the building? What is for example the effect of the H, B and L of the building? By changing these parameters a reduction of the damage can be achieved. From this it can be concluded which parameters must be changed to make a more blast resistant building. Conclusions and recommendations about the building design follows from this.

By changing the parameters of the building one can obtain a building that is better resistant for a shock wave. By changing the parameters in the response aspects discussed in chapter 4.2 the load on the main load bearing structure can be lowered. The three mechanical characteristics that have to be changed are the static strength, natural period and ductility.

An overview of the parameters used in the methods is given in these tables. For each parameter also is given if it should be maximized or minimized for the benefit of or to achieve a more blast resistant building.

Volume tank truck	V	[m^3]	Minimize
Distance	r	[m]	Maximize
Liquid density	ρ	[kg/m^3]	Minimize
Atmospheric pressure	P_0	[kPa]	Minimize

Table 4-7: Parameter optimization load

Peak overpressure at building	P_s	[kPa]	Minimize
Reflected peak overpressure at building	P_r	[kPa]	Minimize
Reflection coefficient	rc	[-]	Minimize
Angle of impact	α	[°]	Maximize
Atmospheric pressure	P_0	[kPa]	Maximize
Drag coefficient	C_D	[-]	Minimize
Dynamic pressure	Q_D	[kPa]	Minimize
Smallest value of H and 1/2*B of building	S	[m]	Minimize
Time at which $P_r = P_s + Q_D$	t_s	[ms]	Minimize
Positive phase duration at building	t_p	[ms]	Minimize
Impulse	I	[kPa*s]	Minimize

Height of the building	H	[m]	Maximize
Width of the building	B	[m]	Maximize
Speed of wave front	U	[m/s]	Maximize
Width of glass	a	[m]	Minimize
Length of glass	b	[m]	Minimize

Table 4-8: Parameter optimization aspects

Static strength	P_{st}	[kPa]	Maximize
Height of building	H	[m]	Maximize
Safety factor	β	[-]	Maximize
Natural frequency	f	[s^{-1}]	Minimize
Natural period	T	[s]	Maximize
Angular rotation	ω	[s^{-1}]	Minimize
Building length	L	[-]	Minimize
Number of floors	n	[-]	Maximize
Specific mass	ρ	[kg/m^3]	Maximize
Beam length	B_k	[m]	Maximize
Column width	b_k	[m]	Minimize
Modulus of elasticity of material structure	E	[Pa]	Minimize
Number of columns	n_k	[-]	Minimize
Column height	h	[m]	Maximize
Maximum elastic deformation	$\hat{x}_{el}$	[m]	Minimize
Maximum plastic deformation	$\hat{x}$	[m]	Maximize

Table 4-9: Parameter optimization mechanical properties

Width of glass	a	[m]	Minimize
Length of glass	b	[m]	Minimize
Thickness of glass	d	[m]	Maximize
Modulus of elasticity of glass	E	[m^4]	Maximize
Height of building	H	[m]	Maximize
Safety factor	β	[-]	Maximize
Width of core	b	[m]	Maximize
Length of core	h	[m]	Maximize
Story height	S	[m]	Minimize
Length of beam	l	[m]	Minimize
Width of beam	b	[m]	Maximize
Height of beam	h	[m]	Maximize
Reinforcement steel	A_s	[m^2]	Maximize
Length of column	l_c	[m]	Minimize
Width of column	b_c	[m]	Maximize
Height of column	h_c	[m]	Maximize

Table 4-10: Parameter optimization mechanical properties elements

A high building is designed with a larger horizontal design wind load than a low building which results in a larger static strength which makes it better resistant against an explosion load. The natural period can be made larger by choosing a higher building, smaller depth and more floors. The natural frequency must be as small as possible. So the more the building can deflect the better. This can be obtained by minimizing the modulus of elasticity and the moment of inertia of the columns and by maximizing the beam length and cross section. A larger ductility can be obtained by applying materials with a large plastic deformation course. The maximum deformation has to be as large as possible.

In some methods a maximization is preferable, in others a minimization. A lower atmospheric pressure is preferred when determining the side on overpressure. When determining the peak reflected overpressure or the dynamic load a higher atmospheric pressure is preferred. Width and height of columns can be maximized or minimized. For the static strength to be large the mass moment of inertia should be as high is possible, so the width and height of the columns cross section should be large. But for the stresses in the column due to the normal load to be as large as possible the cross section should be as small as possible. When the equation for the static strength is further worked out it turns out to be preferable to maximize the cross section of the column.

$$P_{st} = \left(f_b' \cdot \beta + \tfrac{N}{A}\right) \cdot \frac{4 \cdot b^2 \cdot h}{3 \cdot W \cdot l^2} = \frac{1}{3 \cdot W \cdot l^2}\left(f_b' \cdot \beta \cdot 4 \cdot b^2 \cdot h + N \cdot 4 \cdot b\right)$$

Again a contradiction occurs with the length of the beam. To have a low natural frequency a long beam is preferred but to have large static strength needs a short beam.

In conclusion: the building should be as high as possible. To maximize the vibration time the deflections should be as large as possible. So the structure should not be stiff. But the building also should have a large static strength.

In chapter five a quantitative relations between the parameters mentioned here and the outcome of the different methods is given.

4.5.2 Discussion on validation of selected method

In this chapter it is discussed what the quality of the used methods are. For some methods there is some uncertainty about the validity of the outcome or its use and quality in general. These uncertainties and reliabilities are mentioned here. Possibly there are more elaborate calculations needed to provide a better reliability and accuracy than achieved with the methods used in this study. If the sensitivity analysis shows that the parameter following from a certain method has very little influence than a very accurate fine method to determine this parameter is not necessary. A validation of methods to determine the response aspects, static strength, natural frequency, ductility and the response of the elements is given below.

Static strength

The global static strength of the building is determined with help of the governing wind load. It is assumed that this way to determine the static strength is valid or accurate enough.

Natural frequency

In order to make the equivalent static load as small as possible the natural frequency must be minimized. So the natural period T must be as large as possible. When the most rough method is used this means that the building must be as high as possible.

The Master thesis of Van der Meer [18] states a one-mass-spring-system (or SDOF, Single Degree Of Freedom) to determine the static strength and natural frequency are too conservative and a continue response method of the structure is needed. Such a continue response method takes all frequencies of the building into account, not just the lowest frequency. And with a continue response method the building can deform in more than just the deflection in the top of the building and the mass distribution of the building is also taken into account. It is concluded in the study that a SDOF is too conservative and a continue response method should be used. With conservative is meant that the calculated frequency is too high. This study will work with a SDOF because it is easier to work with in this more broad approach of the problem. Further study of the natural frequency *with* a continue response method is recommended.

Ductility

There are no available methods to determine the ductility with a high reliability. A more accurate calculation of the ductility is not needed because its influence is not large, see formulas.

$$\bar{i} = \sqrt{2 \cdot Du - 1}$$

$$\overline{P} = \frac{Du - \frac{1}{2}}{Du}$$

Static strength elements

For the static strength of the core, beams and columns several methods are discussed, the wind load method and the method which uses the maximum stress in concrete or reinforcement bar. Which one of these methods gives a more reliable outcome is recommended for further study.

In order to prescribe the structure response one often uses a Finite Element Method, this is a (computer) program designed for static analysis, but is also used for dynamic response. Whit this method the structure is divided into a large number of finite elements, which are characterized by a number of nodes to which a finite number of degrees of freedom are assigned. The program defines a stiffness matrix which describes the static behaviour of the whole structure. There are several commercial methods available to describe the behaviour of a structure under a specific load. One of them is the FEM (Finite Element Method) TNO DIANA. Originally developed for static loads and vibrations, TNO is now investigating the possibilities to use it for dynamic loads such as an explosion. In a further study to describe the response of the building with more reliability such a FEM is recommended.

Ductility elements

The method used to determine the ductility of the beams would imply that if a beam is symmetrically reinforced the ductility would be infinite. This is not possible, which doubts the use of this method.

Conclusion

It is concluded that the methods used in this study are useful for a simple general approach of the problem but that further study is recommended to develop finer, elaborate methods to confirm this and give more reliable outcomes.

5 Case study

In this chapter the response of a model building under the explosion load is calculated. The structure of the chapter is the same as of chapter 4. For this calculation a model building is defined after which all the methods defined in chapter 4 are used to determine the response, mechanical properties and damage of the building.

5.1 Model building

The model building is a standard office building with a concrete core and a concrete framework structure. Its 35 m in height, has 10 floors and has floor plan of 20 x 20 m. The columns are 3,5 m high and have a cross section of bxh is 0,3x0,3 m^2. The specific mass of the building is 200 kg/m^3. Concrete of B35 has a modulus of elasticity of $31*10^9$ m^4. The floor height is 3,5 m. The beams are 4 m long and have cross section of bxh is 0,2x0,3 m^2, the beams are incorporated in the floors that have a thickness of 0,3 m. The tensile reinforcement percentage ω_0 for B35 and FeB 500 is 1,5 %. All values are chosen random; it is not an existing building. The windows in the building are 1x1,5 m^2 and there are 10 windows per side per floor. The windows are 6 mm thick.

The requirements for the building is that it will not collapse

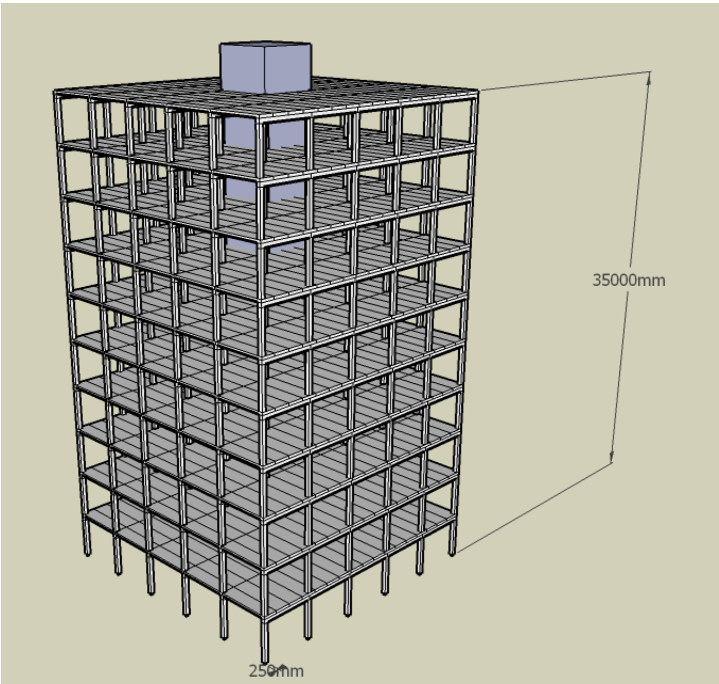


Figure 5-1: Model building

5.2 Response modelling case

5.2.1 Reflection

The perpendicular reflected peak overpressure is given by the following formula.

$$P_r = 2 \cdot P_s + \frac{2,4 \cdot P_s^2}{0,4 \cdot P_s + 2,8 \cdot P_0}$$

Where:

P_r	= Reflected peak overpressure	[kPa]
P_s	= Side on overpressure	[kPa]
P_0	= Atmospheric pressure	[kPa]

The previously found values for P_s can now be implemented.

For P_s = 52 kPa:

$$P_r = 2 \cdot P_s + \frac{2{,}4 \cdot P_s^2}{0{,}4 \cdot P_s + 2{,}8 \cdot P_0} = 2 \cdot 52 + \frac{2{,}4 \cdot 52^2}{0{,}4 \cdot 52 + 2{,}8 \cdot 100} = 126\,\text{kPa}$$

$$rc = \frac{P_r}{P_s} = \frac{126}{52} = 2{,}42$$

5.2.2 Dynamic blast load

The dynamic blast load is

$$Q_D = C_D \cdot \frac{5}{2} \cdot \frac{P_s^2}{7 \cdot P_0 + P_s}$$

Where:

Q_D	= Dynamic blast load	[kPa]
C_D	= Drag coefficient	[-]
P_s	= Peak overpressure	[kPa]
P_0	= Atmospheric pressure	[kPa]

C_D is the drag-coefficient which depends on the shape of the structure. For side on reflection on cube-like structures C_D = 1,05.

$$Q_D = 1{,}05 \cdot \frac{5}{2} \cdot \frac{52^2}{7 \cdot 100 + 52} = 9{,}4\,\text{kPa}$$

5.2.3 Pressure relief wave

The time t_s in which the reflected overpressure is equal to the incoming overpressure plus the dynamic pressure is:

$$t_s = \frac{3S}{U}$$

The speed of the wave front is:

$$U = c_0 \sqrt{1 + \frac{6P_s}{7P_0}}$$

Where:

S	= Smallest value of H and ½*B	[m]
H	= Height of the building	[m]
B	= Width of the building	[m]
U	= Speed of wave front	[m/s]
c_0	= Speed of sound at atmospheric pressure	(=+340 m/s)

When the building is 20 m wide and 35 m high than S = 10 m.

$$U = c_0 \sqrt{1 + \frac{6P_s}{7P_0}} = 340 \cdot \sqrt{1 + \frac{6 \cdot 52}{7 \cdot 100}} = 408{,}9 \text{ m/s}$$

$$t_s = \frac{3S}{U} = \frac{3 \cdot 10}{408{,}9} = 73 \text{ ms}$$

The horizontal load against time as a result of the overpressure, the reflection and the dynamic load can now be captured in the following diagram. t_s is larger than t_p so there will not be a pressure relief wave.

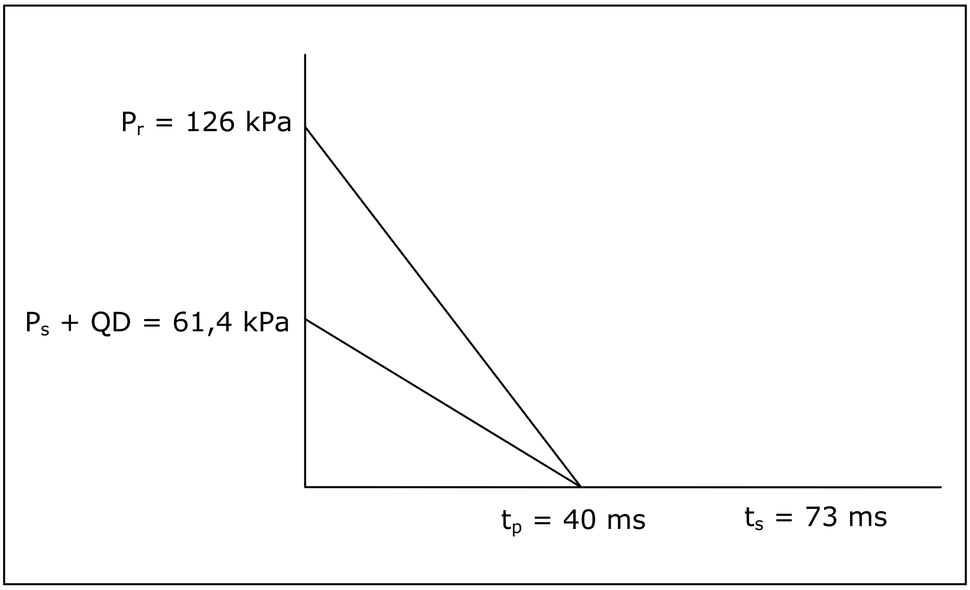


Figure 5-2: Schematic reproduction of the pressure-time curve

The impulse can now be calculated by determining the surface under the graph.

$$I_s = \frac{1}{2} \cdot t_p \cdot P_r = \frac{1}{2} \cdot 0{,}040 \cdot 126 = 2{,}52 \text{ kPa} \cdot \text{s}$$

The impulse that acts on the building is 2,52 kPa*s.

5.2.4 Adjacent buildings

In the described situation in this study there is also dense and high building along the road. However the geometry in this study is different; the effect of an explosion between buildings is investigated and not the effect of one building blocking the other. Therefore the results of the studies described are not used. In stead of using the described factor five and factor three this study will use the methods to determine the reflection used in the previous paragraph on reflection.

5.2.5 Distribution blast on façade of building

In this paragraph the reflected peak overpressure and the impulse on a vertical plane parallel to the road is determined at 20 m distance from the source for a grid of five by five meter. The calculation of the overpressure and impulse is done in the same manner as in chapter 3 and paragraph 5.2.1 to 5.2.3.

$$P_r = 2 \cdot P_s + \frac{2,4 \cdot P_s^2}{0,4 \cdot P_s + 2,8 \cdot P_0}$$

$$rc = \frac{P_r}{P_s}$$

The difference is that now the distance and angle to the source are taken into account. A larger distance and larger impact angle result in a lower reflected peak overpressure en reflection coefficient. For every grid-point the distance, scaled distance and side on overpressure is determined. Than the peak reflected overpressure and the reflection coefficient can be determined. According to the calculated reflected overpressure the rc ranges from 2,0 to 2,4. The rc however decreases at larger distances according to figure 5-3.

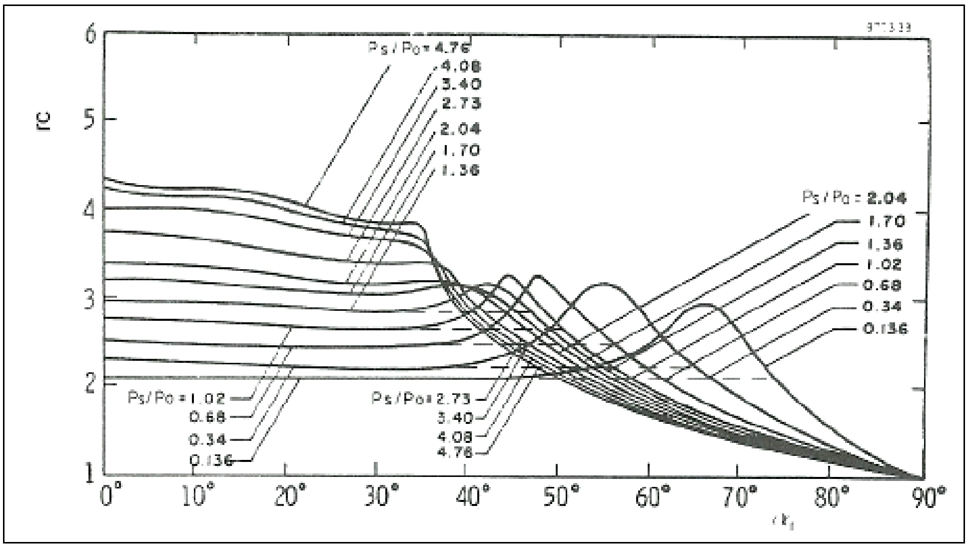


Figure 5-3: Angle of incidence vs. reflection coefficient [7].

An rc of 2,0-2,1 remains constant until an impact angle of 74 degrees. An rc of 2,2-2,4 remains constant till 65 degrees. For a plane 20 m from the source these angles correspond with a distance of 70 m and 47 m respectively. The distances in the chosen grid are larger than this and therefore the rc is adjusted. The range of rc is now from 1,4 to 2,4, which results in a smaller reflected overpressure.

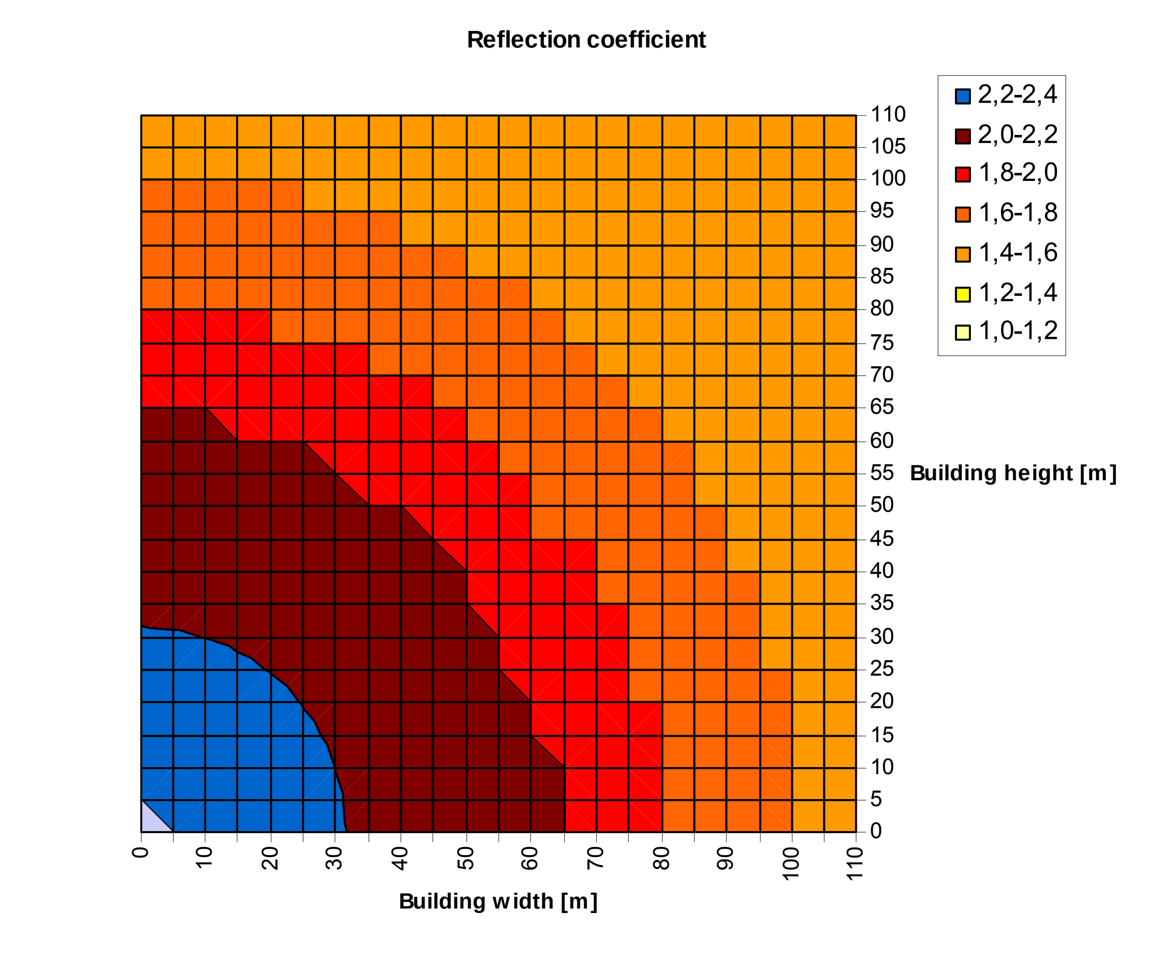


Figure 5-4: Reflection coefficient

The reflected overpressure is presented in the following diagram. The maximum value of the reflected overpressure is the in chapter 3 determined value of 126 kPa and can also be read from this diagram. Some discontinuities can be distinguished in the graphs. The reason for this is that the reflection coefficient must be read out from diagram.

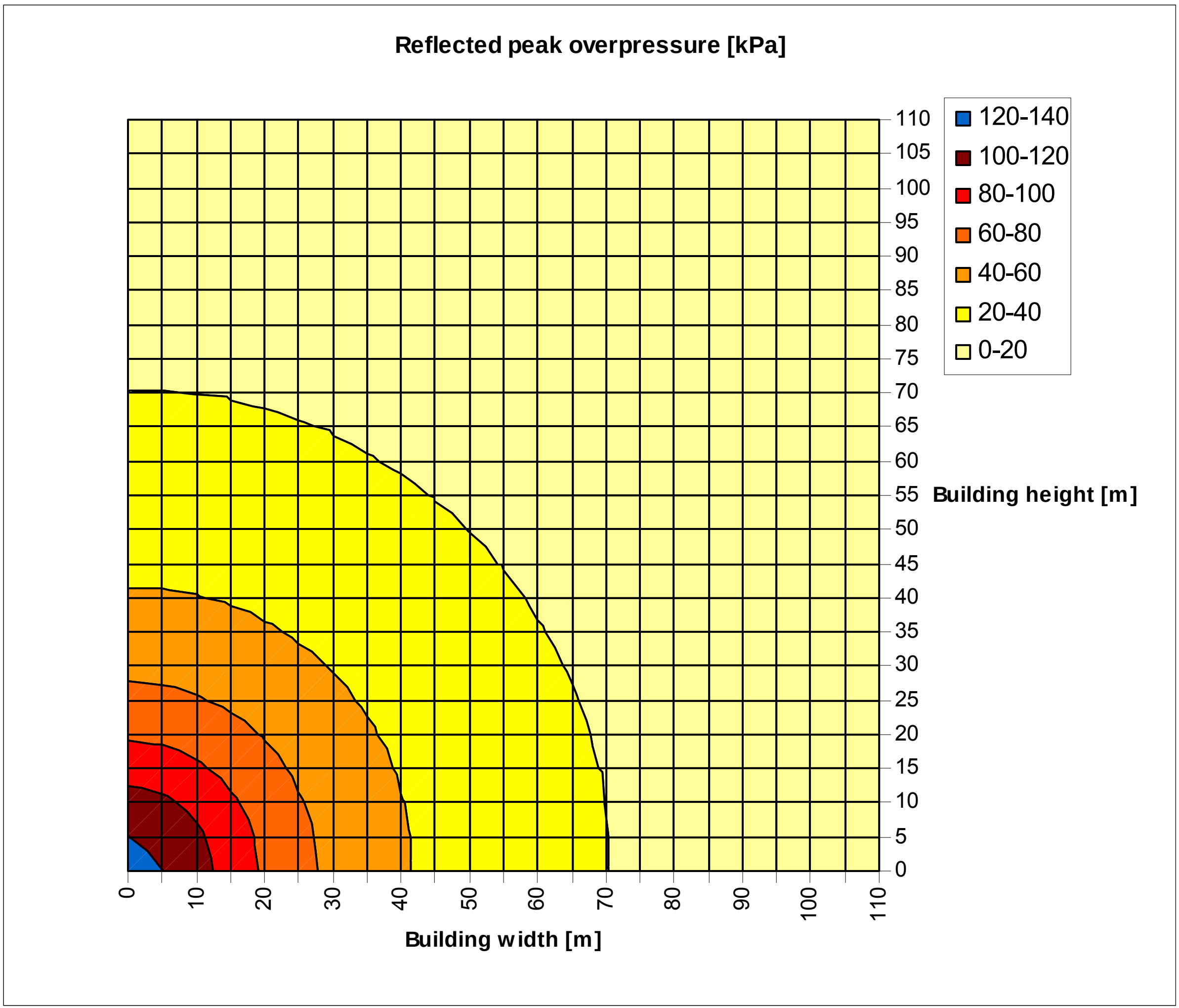


Figure 5-5: Reflected peak overpressure

To determine the impulse the positive phase duration is determined with the method from Van de Berg for every grid point. The resulting impulse for every grid-point is presented in the diagram below.

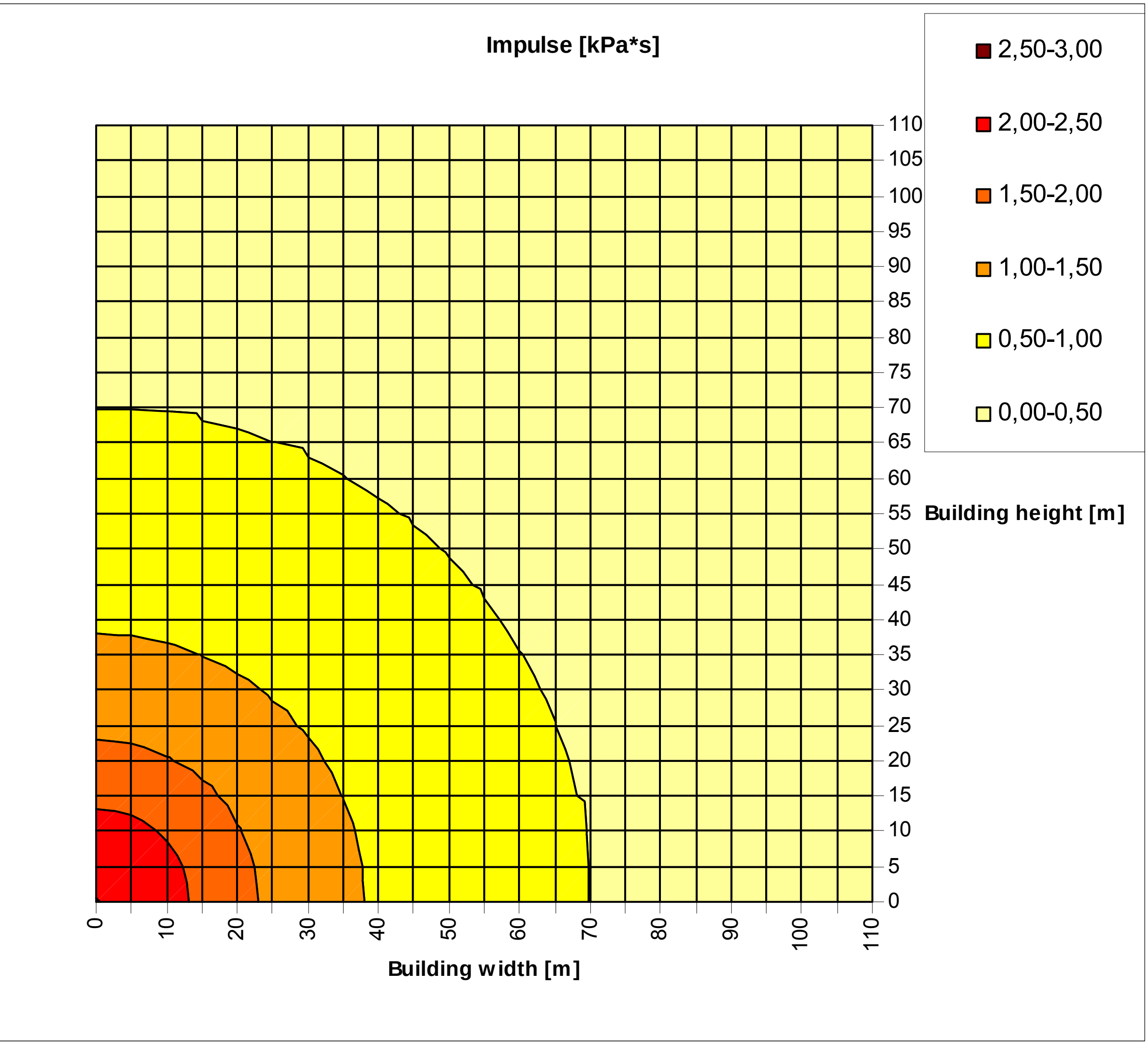


Figure 5-6: Impulse

Note 1:
It is clear from these diagrams that the reflected peak overpressure at the foot of the building is larger than the pressure at the top. For this study a uniform equal distributed load on the building façade is assumed of the maximum P_r = 126 kPa. This is a very conservative approach. It is unknown how the building will react to the non-uniform load. Further study is recommended.

Note 2:
If the road is deepened and the buildings first floor starts 5 m above the road the maximum peak reflected overpressure and the total load at the building will be lower. What the profit is of a deepened road is unknown and recommend for further study.

5.2.6 Round building vs. rectangular building

A round building will have different impact angles than a rectangular building, which will result in a different pressure acting on the building. This difference is calculated in this paragraph. To find out what the reduction of the rc will be for a round building in comparison with a square building the rectangular model building of 20 x 20 m is used and a round building with a radius of 10 m. Three buildings of each are placed beside the road with a distance between them of 10 m. The explosion takes place in front of the first two buildings. The average rc for both square and round building is calculated. The same is done for the second and third building.

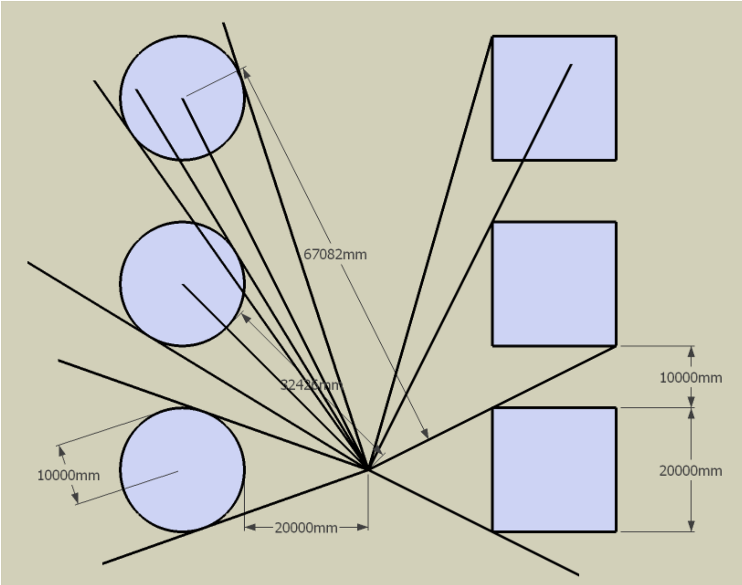


Figure 5-7: Round building vs. rectangular building

The rc of a building can be calculated with:

$$rc = \frac{P_r}{P_s}$$

Where:
P_r = Perpendicular reflected overpressure [kPa]
P_s = Side on overpressure [kPa]

The methods to calculate P_r and P_s are discussed in paragraphs 5.2.1 and 3.2.4. After the impact angles are calculated with standard goniometry the reduced rc can be read from figure 5-3. This results in the following values. The average rc of the rectangular building 1 is 2,4. The rc of building 2 is 2,2 and building 3 is 2,0. Round building 1 and 2 have an average rc of 2,2. The third round building has an rc of 2,3.

The reduction of the reflection by using a round building instead of a square one of 20 m wide and 20 m from the explosion is:

Building 1: $\frac{2,2-2,4}{2,4} \cdot 100 = \text{-8 \%}$

Building 2: $\frac{2,2-2,2}{2,2} \cdot 100 = \text{0 \%}$

Building 3: $\frac{2,3-2,0}{2,0} \cdot 100 = \text{+15 \%}$

On larger distances on the road the rc of a round building will be bigger because the impact angle of a rectangular building decreases with distance and the impact angle of a round building doesn't. A round building always has a part of its surface perpendicular to the incoming wave. But at short distances P_s is much larger than on larger distances.

Also the width of the building facing the blast wave is different for the round and rectangular buildings, the buildings are blocking each other from impact. If P_s and the width of the building facing the blast wave are taken into account the reflected peak overpressure for rectangular and round buildings are:

$$P_r = P_s \cdot rc \cdot B$$

$$P_{r,20,rec} = 52 \cdot 2,4 \cdot 20 = \text{ 2,5 MN/m}$$
$$P_{r,32.4,rec} = 29 \cdot 2,2 \cdot 28,3 = \text{ 1,8 MN/m}$$
$$P_{r,67.0,rec} = 12 \cdot 2,0 \cdot 13,5 = \text{ 0,3 MN/m}$$

$$P_{r,20,round} = 52 \cdot 2,2 \cdot 18,9 = \text{ 2,1 MN/m}$$
$$P_{r,32.4,round} = 29 \cdot 2,2 \cdot 19,3 = \text{ 1,2 MN/m}$$
$$P_{r,67.0,round} = 12 \cdot 2,3 \cdot 15,5 = \text{ 0,4 MN/m}$$

If you take into consideration the distance from the explosion, the shape of the building, the angle of impact and the width of the building than the reduction of the peak reflected overpressure for a round building in comparison to a rectangular building is:

$$\text{Building 1 (20 m): } \frac{2,1\text{-}2,5}{2,5} \cdot 100 = -16 \text{ \%}$$
$$\text{Building 2 (32,3 m): } \frac{1,2\text{-}1,8}{1,8} \cdot 100 = -33 \text{ \%}$$
$$\text{Building 3 (67,0 m):} \frac{0,4\text{-}0,3}{0,3} \cdot 100 = +33 \text{ \%}$$

It can be concluded that the peak reflected overpressure on a round building is 16 % lower than on a comparable rectangular building when an explosion takes place in front of the building and 33 % lower if the explosion takes place in front of the neighbouring building. At larger distances the overpressure on a round building will be higher but the absolute pressure is more than 5 times lower than on closer buildings.

Note 1:
The height of the building is not taken into account which could make a difference. The rc and distance at height also determine the load on the building. Further study is recommended.

Note 2:
The surface area of the round building is lower than that of the square building which is a disadvantage for the round building. Because the larger the surface the larger the load.

5.2.7 Glass failure

Because it is assumed that the glass fails instantly the peak reflected overpressure on the total building will be lower. For the model building the surface area of glass on one side of the building is 150 m^2. The total surface area is 700 m^2.

$$P'_r = \frac{1}{A_{total}}\left(\left(A_{total} - A_{glass}\right) \cdot P_r + P_{st,glass} \cdot A_{glass}\right) = \frac{1}{700} \cdot \left(\left(700 - 150\right) \cdot 126 + 10,5 \cdot 150\right) = 101 \text{ kPa}$$

$$I'_s = \frac{A_{total} - A_{glass}}{A_{total}} \cdot I_s = \frac{700-150}{700} \cdot 2{,}52 = 1{,}98\,\mathrm{kPa} \cdot \mathrm{s}$$

Where:

P'_r	= Corrected peak reflected overpressure	[kPa]
A_{total}	= Total surface of facade	[m^2]
A_{glass}	= Surface of glass in façade	[m^2]
P_r	= Peak reflected overpressure	[kPa]
$P_{st,glass}$	= Static strength glass	[kPa]
I'_s	= Corrected impulse	[kPa*s]
I_s	= Impulse	[kPa*s]

The calculation of the static strength is done in chapter 5.3. The reflected overpressure is reduced from 126 kPa to 101 kPa. The more glass is used in the building the more this reduction will be. This study will work with a reflected peak overpressure of 101 kPa.

5.3 Mechanical properties case

The parameters that are of importance for the structural response of the building are:

For the load
- The shape; shock wave or pressure wave
- Peak overpressure
- Positive phase duration

For the structure
- Static strength
- Natural frequency
- Ductility

The shape of the explosion load is a shock wave. When the shockwave reaches the building the side on overpressure is 52 kPa and the reflected peak overpressure is 126 kPa. These loads are considered to be uniform over the surface of the building, which in reality isn't the case. The positive phase duration is 40 ms. The impulse I is 2,52 kPa*s.

This paragraph determines how much load a building can take. Subsequently the three structural characteristics depicted above, static strength, natural frequency and ductility are worked out. Also the properties of structural elements are determined.

5.3.1 Static strength

The load on the building depends on the height. For a building with a height of 35 m in wind area II a pressure at the top of the building of 1,3 kPa applies. The foot of the building has a pressure of 0,54 kPa.

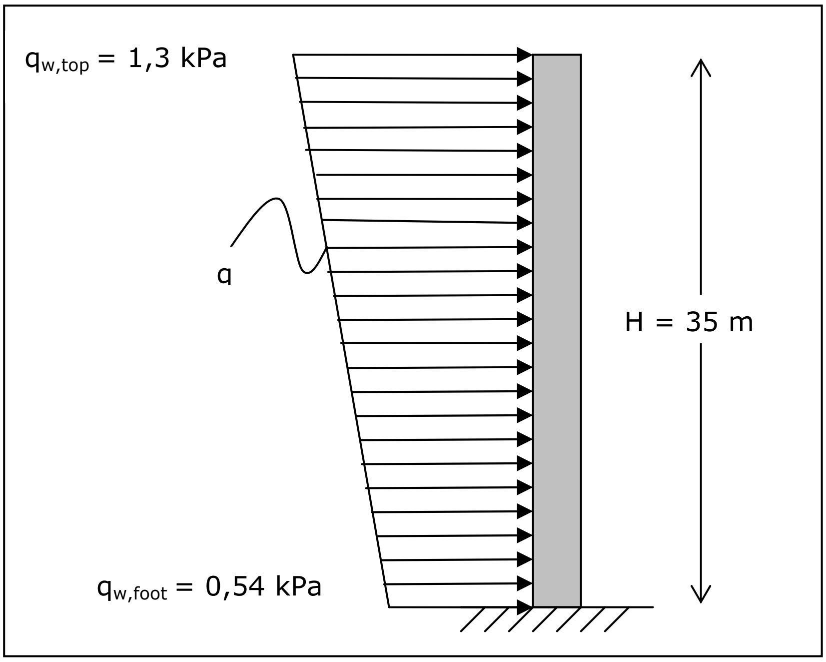


Figure 5-8: Schematization wind load on a 35 m high building

The fixed moment at the foot of the building is:

$$M_{w,core} = C_w \cdot \left(\frac{1}{2} \cdot q_{w,foot} \cdot H^2 + \frac{1}{3}\left(q_{w,top} - q_{w,foot}\right) \cdot H^2 \right)$$
$$= 1{,}8 \cdot \left(\frac{1}{2} \cdot 0{,}54 \cdot 35^2 + \frac{1}{3}(1{,}3 - 0{,}54) \cdot 35^2 \right) = 1154\,\mathrm{kPa} \cdot \mathrm{m}^2$$

The corresponding static strength is now:

$$P_{st} = \frac{2}{H^2} \cdot \beta \cdot M_w$$
$$= \frac{2}{35^2} \cdot 2{,}7 \cdot 1154 = 5{,}1\,\mathrm{kPa}$$

5.3.2 Natural frequency

The four described empirical methods in chapter 4.3 result in the following values for the natural period.

- $$T = k_1 \cdot H$$ $$= 0{,}02 \cdot 35 = 0{,}70\ \mathrm{s}$$
- $$T = k_2 \cdot \frac{H}{\sqrt{L}} = 0{,}09 \cdot \frac{35}{\sqrt{20}} = 0{,}70\ \mathrm{s}$$
- $$T = 0{,}1 \cdot n = 0{,}1 \cdot 10 = 1{,}00\ \mathrm{s}$$
- $$T = k_3 \cdot H^{\frac{3}{4}} = 0{,}061 \cdot 35^{\frac{3}{4}} = 0{,}88\ \mathrm{s}$$

A more elaborate method to determine the natural period is presented by Raleigh.

$$T = \sqrt{\frac{0,25}{\delta}}$$

Where δ is the maximum deformation and consists of shear deformation and bending deformation.

The moment of elasticity of concrete is $31*10^9$ Pa. The specific mass of an office building is 200 kg/m^3. The dimension of the model building are number of floors n =10, column height h = 3m, distance between columns is B_k = 4m, Length of the building L = 20 m. In the model building there are in the direction of the load five columns after each other, so n_k = 5.

The mass moment of inertia of five consecutive columns is:

$$\Sigma I_k = n_k \cdot \frac{1}{12} \cdot b_k^4 = 5 \cdot \frac{1}{12} \cdot 0,5^4 = 3,375 \cdot 10^{-3} \text{ m}^4$$

The dead load on one row of columns in the direction of the load is:

$$\begin{aligned} p_{e.g.} &= \rho \cdot g \cdot B_k \cdot L \\ &= 200 \cdot 10 \cdot 4 \cdot 20 = 1,6 \cdot 10^5 \text{ N/m} \end{aligned}$$

The deformation of a column as a result of shear is:

$$\begin{aligned} \delta &= \frac{p_{e.g.} \cdot n^2 \cdot h^2}{24 \cdot E \cdot \Sigma I_k} \\ &= \frac{1,6 \cdot 10^5 \cdot 10^2 \cdot 3,5^2}{24 \cdot 31 \cdot 10^9 \cdot 3,375 \cdot 10^{-3}} = 78{,}1 \text{ mm} \end{aligned}$$

The deformation of a column as a result of bending is:

$$\begin{aligned} \delta &= \frac{p_{e.g.} \cdot n^4 \cdot h^4}{4 \cdot E \cdot A_k \cdot L^2} \\ &= \frac{1,6 \cdot 10^5 \cdot 10^4 \cdot 3,5^4}{4 \cdot 31 \cdot 10^9 \cdot 0,3^2 \cdot 20^2} = 53,8 \text{ mm} \end{aligned}$$

$$\delta_{\max} = \delta_{shear} + \delta_{bending} = 0,0781 + 0,0538 = 0,1319 \text{ m}$$

$$f = \sqrt{\frac{0,25}{\delta}} = \sqrt{\frac{0,25}{0,1319}} = 1,38 \text{ s}^{-1}$$

$$T = \frac{1}{f} = \frac{1}{1,38} = 0,72 \text{ s}$$

The values of the natural frequency that are found here differ only a little from each other, 0.70, 0.70, 1.00, 0.88 and 0.72 respectively. Also the last more elaborate method resembles the values of the empirical methods.

The following graph shows the natural frequency for the five different methods versus the height of the building.

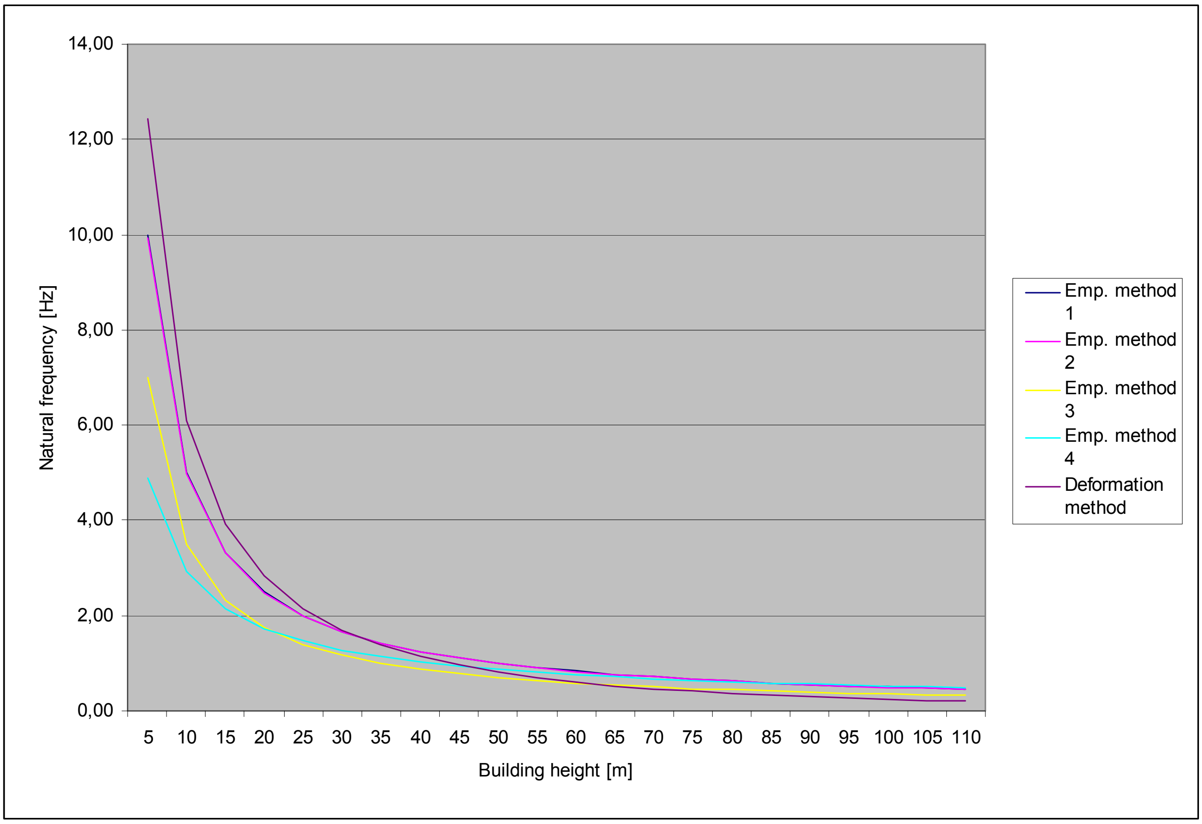


Figure 5-9: Natural frequency versus the building height for different methods.

In the following graph a line is added that gives the natural frequency of the deformation method when columns with a width of 0,6 m are used. It shows the deformation method is very sensitive for this value.

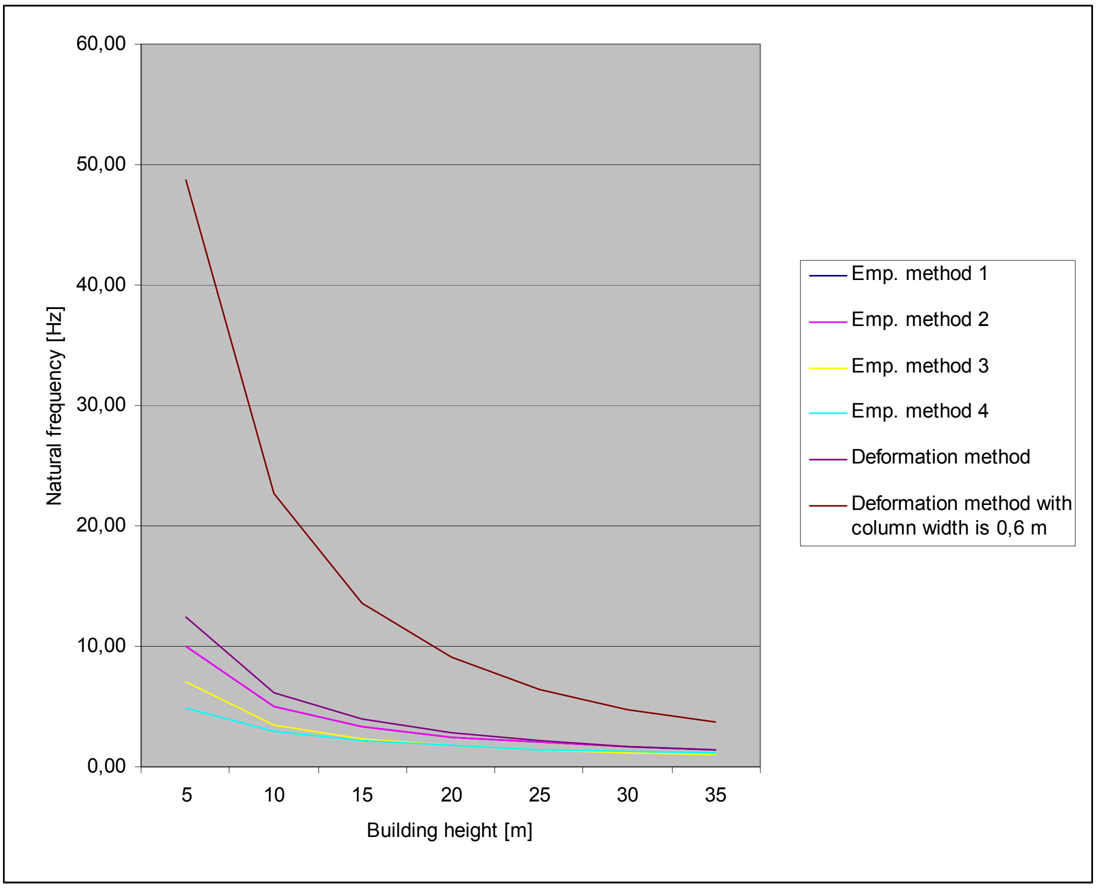


Figure 5-10: Natural frequency versus the building height for different methods.

With the deformation method the natural frequency of a 35 m high building ranges from 0,2 Hz for a column of 0,1 m wide and 106 Hz for a column of 0,8 wide. These values are not reasonable. Only the largest skyscrapers have a natural frequency of 0,1 Hz and the natural frequency is in general not larger than 10 Hz. Because of this high sensitivity to the value of the column width it is chosen not to work with the deformation method but with empirical method 1.

For the model building a natural period of 0,70 s is worked with. The natural frequency and the angular rotation are:

$$f = \frac{1}{T} = \frac{1}{0{,}70} = 1{,}43\,\mathrm{s}^{-1}$$

$$\omega = f \cdot 2\pi = 8{,}98\,\mathrm{s}^{-1}$$

5.3.3 Ductility

According to [7] the ductility of a concrete building generally is about 4. This is global average value. The maximum ductility is about 10. In this study a ductility of 4 is assumed.

5.3.4 Mechanical properties elements

<u>Static strength of elements</u>

Static strength glass

For a window of 1 x 1,5 m and 6 mm thick this means.

a	= Smallest dimension of window	= 1,0 m
b	= Largest dimension of window	= 1,5 m
d	= Thickness of window	= 0,006 m
v	= Poisson ratio	= 0,25 m
E	= Modulus of elasticity	= 75*10^9 Pa
ρ	= Dead load of material	= 2500 kg/m^3
C_7	= Constant	= 1 m
C_8	= Constant	= 14,9 MPa

$$f_t = C_8 \cdot \left(\frac{d}{C_7}\right)^{-0,32} \cdot \left(\frac{b}{a}\right)^{0,47}$$

$$f_t = 14,9 \cdot 10^6 \cdot \left(\frac{6 \cdot 10^{-3}}{1}\right)^{-0,32} \cdot \left(\frac{1,5}{1}\right)^{0,47} = 92,67 \cdot 10^6 \text{ Pa}$$

$$\alpha = \frac{16}{\pi^6 \cdot \left(1 + \left(\frac{a}{b}\right)^2\right)^2}$$

$$\alpha = \frac{16}{\pi^6 \cdot \left(1 + \left(\frac{1}{1,5}\right)^2\right)^2} = 7,98 \cdot 10^{-3} \text{ [-]}$$

$$q_{middle} = \frac{f_t}{6\pi^2 \cdot \alpha \cdot \left(\frac{a}{d}\right)^2 \cdot \left(1 + \nu\left(\frac{a}{b}\right)^2\right)}$$

$$q_{middle} = \frac{92,67 \cdot 10^6}{6\pi^2 \cdot 7,98 \cdot 10^{-3} \cdot \left(\frac{1}{6 \cdot 10^{-3}}\right)^2 \cdot \left(1 + 0,25 \cdot \left(\frac{1}{1,5}\right)^2\right)} = 6354 \text{ Pa}$$

$$q_{corner} = \frac{f_t}{6\pi^2 \cdot \alpha \cdot \left(\frac{a^3}{d^2 \cdot b}\right) \cdot (1 - \nu)}$$

$$q_{hoek} = \frac{92,67 \cdot 10^6}{6\pi^2 \cdot 7,98 \cdot 10^{-3} \cdot \left(\frac{1^3}{\left(6 \cdot 10^{-3}\right)^2 \cdot 1,5}\right) \cdot (1 - 0,25)} = 14119 \text{ Pa}$$

$$\delta_{kr} = 6 \cdot \left(\frac{b}{a}\right)^{\frac{3}{2}} \cdot d$$

$$\delta_{kr} = 6 \cdot \left(\frac{1,5}{1}\right)^{\frac{3}{2}} \cdot 6 \cdot 10^{-3} = 66 \cdot 10^{-3} \text{ m}$$

$$D = \frac{E \cdot d^3}{12\left(1-v^2\right)}$$

$$D = \frac{75 \cdot 10^9 \cdot \left(6 \cdot 10^{-3}\right)^3}{12\left(1-0,25^2\right)} = 1440 \text{ Nm}$$

The deformation is:

$$\delta\left(q\right) = \frac{\alpha \cdot q_{middle} \cdot a^4}{D}$$

$$\delta\left(q\right) = \frac{7,98 \cdot 10^{-3} \cdot 6354 \cdot 1^4}{1440} = 35 \cdot 10^{-3} \text{ m}$$

The deflection in the middle is smaller than the critical deflection so the glass will fail in the middle. The failure pressure is:

$$P_{st,glass} = q_{middle} + \frac{\delta\left(q_{middle}\right)}{\delta_{kr}} \cdot \left(q_{corner} - q_{middle}\right) = 6354 + \frac{35 \cdot 10^{-3}}{66 \cdot 10^{-3}} \cdot \left(14119 - 6354\right) = 10,5 \text{ kPa}$$

Static strength concrete core

Wind load

The governing design load on the core is the wind load. This wind load results in a moment which is the same as for the global building.

C_w depends on the friction and suction that can occur and is maximum 1,8. For this study this value of 1,8 is used.

$$M_{w,\ker n} = B \cdot C_w \cdot \left(\frac{1}{2} \cdot q_{w,foot} \cdot H^2 + \frac{1}{3}\left(q_{w,top} - q_{w,foot}\right) \cdot H^2\right)$$

$$= 20 \cdot 1,8 \cdot \left(\frac{1}{2} \cdot 0,54 \cdot 35^2 + \frac{1}{3}\left(1,3 - 0,54\right) \cdot 35^2\right) = 23079 \text{ kPa} \cdot \text{m}^3 = 23079 \text{ kNm}$$

$$P_{st,core,wind} = \frac{1}{B} \cdot \frac{2}{H^2} \cdot \beta \cdot M_w$$

$$= \frac{1}{20} \cdot \frac{2}{35^2} \cdot 2,7 \cdot 23079 = 5,1 \text{ kPa}$$

Stresses in the core

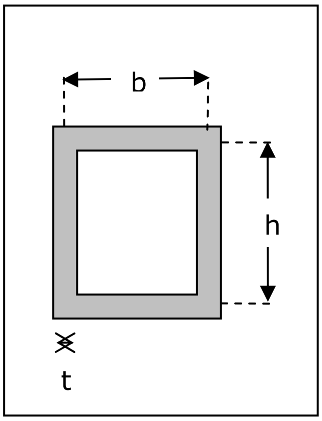


Figure 5-11: Schematization square concrete core

The dimensions of the core are centre to centre 5x5 m and a 30 cm thick wall. Because it is a square core b=h. The mass moment of inertia of a rectangular concrete core is:

$$I_{yy} = \frac{1}{12}\cdot(b+t)^3\cdot(h+t) - \frac{1}{12}\cdot(b-t)^3\cdot(h-t) = \frac{1}{12}\cdot(b+t)^4 - \frac{1}{12}\cdot(b-t)^4$$
$$= \frac{1}{12}\cdot(5{,}3)^4 - \frac{1}{12}\cdot(4{,}7)^4 = 25{,}09\ \mathrm{m}^4$$

Where:

b	= Width of core	[m]
h	= Height of core	[m]
t	= Thickness of wall	[m]

The load of the building that is transferred to the core of the building is assumed to be an area of 10x10 m above the core with a weight of 200 kg/m^3.

$$N = b\cdot h\cdot l\cdot G\cdot g$$
$$= 10\cdot 10\cdot 35\cdot 200\cdot 10 = 7\cdot 10^6\ \mathrm{N}$$

$$A = 0{,}3\cdot 4\cdot 5 = 6{,}0\ \mathrm{m}^2$$

$$f_{e.g.} = \frac{N}{A} = \frac{7\cdot 10^6}{6{,}0\cdot 10^6} = 1{,}17\ \mathrm{N/mm}^2$$

The concrete core will crack if the explosion load results in the maximum tensile stress in the core cross section being exceeded. This maximum tensile stress is f_b = 1,40 N/mm^2 for cracking of B35 concrete. The compression strength f_b' for B35 concrete is 21 N/mm2. These values can be multiplied by the safety factor β. The exceeding of the tensile strength will lead to cracks but not to collapse, the exceeding of the compression strength will lead to collapse. Because there is a compression stress in the core due to the dead weight of the building the maximum stress is:

$$P_{st,core,cracking} = \frac{4\cdot\beta\cdot\left(f_b + f_{e.g.}\right)\cdot I_{yy}}{L^2\cdot b\cdot B} = \frac{4\cdot 2{,}7\cdot(1{,}4+1{,}17)\cdot 10^6\cdot 25{,}09}{35^2\cdot 5\cdot 20} = 5{,}7\ \mathrm{kPa}$$

$$P_{st,core,compression} = \frac{4 \cdot \beta \cdot \left(f'_b - f_{e.g.}\right) \cdot I_{yy}}{L^2 \cdot b \cdot B} = \frac{4 \cdot 2,7 \cdot (21 - 1,17) \cdot 10^6 \cdot 25,09}{35^2 \cdot 5 \cdot 20} = 509 \text{ kPa}$$

Stresses in reinforcement steel

The pressure on the façade of the building that will lead to yielding of the reinforcement in the concrete core is:

$$P_{st,core,yielding} = \frac{8 \cdot \omega_0 \cdot b^2 \cdot t \cdot f_s \cdot \beta}{3 \cdot L \cdot H^2} = \frac{8 \cdot 0,015 \cdot 5^2 \cdot 0,3 \cdot 435 \cdot 10^6 \cdot 2,7}{3 \cdot 20 \cdot 35^2} = 14,4 \text{ kPa}$$

When this static strength is exceeded the building can be considered collapsed.

Shear stresses

When using the wind load method the shear force in the core is:

$$P_{st,core,wind,shear} = C_w \cdot \beta \cdot \left(q_{w,foot} + \frac{1}{2} \cdot \left(q_{w,top} - q_{w,foot}\right)\right)$$

$$= 1,8 \cdot 2,7 \cdot \left(0,54 + \frac{1}{2} \cdot (1,3 - 0,54)\right) = 4,5 \text{ kPa}$$

The static strength of the core based on the stresses in the cross section can be calculated with the following formula. The maximum shear stress in a cross section with shear reinforcement for B35 is 4,2 N/mm^2.

$$P_{st,core,cracking,shear,1} = \frac{\beta \cdot \tau \cdot \left((b+t)^4 - (b-t)^4\right)}{B \cdot H \cdot (b-t) \cdot 3 \cdot b} = \frac{2,7 \cdot 4,2 \cdot 10^6 \cdot \left(5,3^4 - 4,7^4\right)}{20 \cdot 35 \cdot 4,7 \cdot 3 \cdot 5} = 69 \text{ kPa}$$

The maximum shear stress in the other direction is:

$$P_{st,core,cracking,shear,2} = \frac{4 \cdot \beta \cdot \tau \cdot \left((b+t)^4 - \cdot (b-t)^4\right)}{15 \cdot B \cdot H \cdot b} = \frac{4 \cdot 2,7 \cdot 4,2 \cdot 10^6 \cdot \left(5,3^4 - \cdot 4,7^4\right)}{15 \cdot 20 \cdot 35 \cdot 5} = 260 \text{ kPa}$$

The value of 4,2 N/mm^2 is for concrete with shear reinforcement, therefore the reinforcement is going the yield and the core will fail.

Static strength of floors

Wind load

Just as the strength of the global building and the core of the building the static strength of the floors can be calculated using the governing wind load.

The wind pressure q_w depends on the height of the floor in the building. See table in paragraph 4.3.1. The static strength based on the governing wind load is now:

$$P_{st,floor,wind,\max} = C_w \cdot \beta \cdot q_w = 1,8 \cdot 2,7 \cdot 1,3 = 6,3 \text{ kPa}$$

$$P_{st,floor,wind,\min} = C_w \cdot \beta \cdot q_w = 1,8 \cdot 2,7 \cdot 0,54 = 2,6 \text{ kPa}$$

Stresses in floor

In stead of using the governing wind load the static strength can also be determined by calculating the maximum stresses in its cross section.

$$P_{st,floor,cracking} = \frac{4 \cdot f_b \cdot \beta \cdot h_{floor} \cdot B^2}{3 \cdot S \cdot L^2} = \frac{4 \cdot 1{,}4 \cdot 2{,}7 \cdot 300 \cdot 20000^2}{3 \cdot 3500 \cdot 20000^2} = 0{,}432\ \text{N/mm}^2 = 432\ \text{kPa}$$

This static strength represents the point at which the concrete floor will start cracking.

Stresses in reinforcement steel

$$P_{st,floor,yielding} = \frac{8 \cdot h_{floor} \cdot B^2 \cdot \omega_0 \cdot f_s \cdot \beta \cdot 0{,}9}{3 \cdot S \cdot L^2}$$
$$= \frac{8 \cdot 300 \cdot 20000^2 \cdot 0{,}015 \cdot 435 \cdot 2{,}7 \cdot 0{,}9}{3 \cdot 3500 \cdot 20000^2} = 3{,}624\ \text{N/mm}^2 = 3624\ \text{kPa}$$

When this static strength is exceeded the reinforcement in the floor will start to yield. The building can than be considered collapsed.

Shear stresses in floor

When using the wind load method the shear force in the floor is:

$$P_{st,floor,wind,shear} = C_w \cdot q_w \cdot \beta = 1{,}8 \cdot 1{,}3 \cdot 2{,}7 = 6{,}3\ \text{kPa}$$

The maximum shear stress in a rectangular cross sections is:

$$P_{st,floor,stress,shear} = \frac{2 \cdot \tau_{\max} \cdot 2 \cdot h_{floor} \cdot B \cdot \beta}{3 \cdot L \cdot S} = \frac{2 \cdot 4{,}2 \cdot 10^6 \cdot 2 \cdot 0{,}3 \cdot 20 \cdot 2{,}7}{3 \cdot 20 \cdot 3{,}5} = 1296\ \text{kPa}$$

The maximum shear stress in a cross section with shear reinforcement for B35 is 4,2 N/mm^2. Because this is the maximum shear stress *with* reinforcement the core can be considered collapsed when this value is exceeded.

Static strength of columns and walls

Stresses in column

In the columns of the building there is besides the horizontal working load of the explosion also the dead load of the building. But the columns are only loaded when the blast wave has entered the building. The overpressure will push the floors down and the ceiling up. It is assumed that this cancels out the normal force in the columns.

The maximum tensile stress in concrete is 1,4 N/mm2. This value can be multiplied by the safety factor β. When this value is exceeded the concrete column will start to crack.

The static strength based on the tensile stresses in the columns is:

$$P_{st,column,cracking} = \frac{4 \cdot f_b \cdot \beta \cdot d^2}{3 \cdot l^2} = \frac{4 \cdot 1{,}4 \cdot 10^6 \cdot 2{,}7 \cdot 0{,}3^2}{3 \cdot 3{,}5^2} = 37{,}0\ \text{kPa}$$

Stresses in reinforcement steel in column

$$P_{st,column,yielding} = \frac{2 \cdot \omega_0 \cdot f_s \cdot \beta \cdot 0,9 \cdot d^2}{3 \cdot l^2} = \frac{2 \cdot 0,015 \cdot 435 \cdot 10^6 \cdot 2,7 \cdot 0,9 \cdot 0,3^2}{3 \cdot 3,5^2} = 77,7 \text{ kPa}$$

When this static strength is exceeded the reinforcement in the column will start to yield. The building can than be considered collapsed.

Shear stresses in column

The maximum shear stress in a rectangular cross sections is:

$$P_{st,column,stress,shear} = \frac{4 \cdot \tau_{\max} \cdot d \cdot \beta}{3 \cdot l} = \frac{4 \cdot 4,2 \cdot 10^6 \cdot 0,3 \cdot 2,7}{3 \cdot 3,5} = 1296 \text{ kPa}$$

Columns in facade

The factor with which the static strength of a column inside the building can be multiple to get the static strength of a column when it is placed in the façade is the following.

$$c_{facade} = \frac{b_{column}}{l_{beam}} = \frac{0,3}{4} = 0,075$$

The static strength of a column inside the building is 13,3 times larger than a column placed in the façade. When different dimensions are used this factor can range from approximately 10 to 30.

Walls

Shear walls place perpendicular to the load have the same static strength as columns. Walls placed in direction of load have far higher static strength. A shear wall of 5 m in length has the following static strengths.

$$P_{st,column,cracking} = \frac{4 \cdot f_b^{'} \cdot \beta \cdot d^2}{3 \cdot l^2} = \frac{4 \cdot 1,4 \cdot 10^6 \cdot 2,7 \cdot 5^2}{3 \cdot 3,5^2} = 10286 \text{ kPa}$$

$$P_{st,column,yielding} = \frac{2 \cdot \omega_0 \cdot f_s \cdot \beta \cdot 0,9 \cdot d^2}{3 \cdot l^2} = \frac{2 \cdot 0,015 \cdot 435 \cdot 10^6 \cdot 2,7 \cdot 0,9 \cdot 5^2}{3 \cdot 3,5^2} = 21572 \text{ kPa}$$

$$P_{st,column,stress,shear} = \frac{4 \cdot \tau_{\max} \cdot d \cdot \beta}{3 \cdot l} = \frac{4 \cdot 4,2 \cdot 10^6 \cdot 5 \cdot 2,7}{3 \cdot 3,5} = 21600 \text{ kPa}$$

Conclusion strength

The calculated static strengths of the different elements are gathered in this table.

Element	Failure mode		Static strength P_{st} (kPa)
Global building	Bending	Wind load	5,1
Glass	Shear/bending	Brittle cracking glass	10,5
Core	Bending	Wind load on core	5,1
	Bending	Exceeding tensile strength concrete, cracking of core	5,7
	Bending	Exceeding compression strength concrete	509
	Bending	Reinforcement in concrete starts to yield	14,4
	Shear	Wind load resulting in shear failure of core	4,5
	Shear	Exceeding tensile strength, yielding reinforcement in core because of shear failure	69
Floor	Bending	Wind load on floor	2,6
	Bending	Cracking of floor	432
	Bending	Yielding rebar in floor	3624
	Shear	Wind load resulting in shear failure of floor	6,3
	Shear	Exceeding shear stress in floor	1296
Columns and perpendicular walls	Bending	Exceeding tensile strength concrete, cracking of column	37,0
	Bending	Yielding of rebar in column	77,7
	Shear	Exceeding shear stress in column	1296
Wall	Bending	Exceeding tensile strength concrete, cracking of wall	10286
	Bending	Yielding of rebar in wall	21572
	Shear	Exceeding shear stress in wall	21600

Table 5-1: Static strength for a concrete building of 35 m high

From this table it can be read which element and failure mode is governing. The method to determine the static strength of the floor with the wind load is considered to be invalid, because the design load for the floor is not the wind load. For the elements it is assumed that the element will fail or collapse when the reinforcement in the concrete starts to yield. The static strength for the elements is.

$$P_{st,global} = 5,1\ kPa$$

$$P_{st,glass} = 10,5\ kPa$$

$$P_{st,core} = 14,4\ kPa$$

$$P_{st,floor} = 1296\ kPa$$

$$P_{st,column} = 77,7\ kPa$$

$$P_{st,wall} = 21572\ kPa$$

The glass is the first to fail. Of the structural elements the core is the first one to fail. The calculated static strength of the global building is smaller than the static strength of the core. This is because different methods are used. The static strength of the core and the global building are both used in this study. When columns in the façade are used the static strength of the column is reduced to 77,7/13,3=5,8 kPa, which would make the columns much less resistant to a blast and the first to fail. The floors have a very large static strength. When shear walls are placed in perpendicular to the load they show the same static strength as the columns. When the walls are place in the direction of the load they have a very high static strength.

<u>Natural frequency elements</u>

Natural frequency glass

For the model building applies:

a	= Smallest dimension of window	= 1,0 m
b	= Largest dimension of window	= 1,5 m
d	= Thickness of window	= 0,006 m
v	= Poisson ratio	= 0,25 m
E	= Modulus of elasticity	= $75*10^9$ Pa
ρ	= dead load of material	= 2500 kg/m^3

$$I_{yy} = \frac{1}{12} \cdot a \cdot d^3$$

The lowest Eigen frequency for a plate is:

$$f = \frac{\pi}{2}\left(\frac{1}{a^2} + \frac{1}{b^2}\right) \cdot \sqrt{\frac{E \cdot I}{\rho \cdot d \cdot \left(1 - v^2\right)}}$$

$$= \frac{\pi}{2}\left(\frac{1}{1,0^2} + \frac{1}{1,5^2}\right) \cdot \sqrt{\frac{75 \cdot 10^9 \cdot \frac{1}{12} \cdot 1,0 \cdot 0,006^3}{2500 \cdot 0,006 \cdot \left(1 - 0,25^2\right)}} = 22,23 \text{ s}^{-1}$$

$$T = \frac{1}{f} = \frac{1}{22,23} = 0,045 \text{ s}$$

$$\omega = f \cdot 2\pi = 22,23 \cdot 2\pi = 139,68 \text{ s}^{-1}$$

Natural frequency concrete core

The natural frequency of the concrete core is the same as the natural frequency of the whole building.

Natural frequency of floors

a	= Length of the plate	= 20 m
b	= Width of the plate	= 20 m
E	= Modulus of elasticity	= $31*10^9$ Pa
I	= Moment of inertia	= $\frac{1}{12} \cdot B \cdot d^3$
ρ	= Dead weight of the plate	= 2400 kg/m^3

d = Thickness of the plate = 0,3 m
v = Poisson's ratio = 0,2

The natural frequency of plates is given by the following formula [7]:

$$f = \frac{\pi}{2}\cdot\left(\frac{1}{a^2}+\frac{1}{b^2}\right)\cdot\sqrt{\frac{E\cdot I}{\rho\cdot d\cdot\left(1-v^2\right)}} = \frac{\pi}{2}\cdot\left(\frac{1}{a^2}+\frac{1}{b^2}\right)\cdot\sqrt{\frac{E\cdot\frac{1}{12}\cdot B\cdot d^3}{\rho\cdot d\cdot\left(1-v^2\right)}}$$

$$= \frac{\pi}{2}\cdot\left(\frac{2}{a^2}\right)\cdot\sqrt{\frac{E\cdot\frac{1}{12}\cdot B\cdot d^2}{\rho\cdot\left(1-v^2\right)}} = \frac{\pi}{2}\cdot\left(\frac{2}{20^2}\right)\cdot\sqrt{\frac{31\cdot 10^9\cdot\frac{1}{12}\cdot 20\cdot 0,3^2}{2400\cdot\left(1-0,2^2\right)}} = 11,16\ \text{s}^{-1}$$

$$T = \frac{1}{f} = \frac{1}{11,16} = 0,090\ \text{s}$$

$$\omega = f\cdot 2\pi = 11,16\cdot 2\pi = 70,12\ \text{s}^{-1}$$

Natural frequency column

E = Modulus of elasticity = $31*10^9$ Pa
I = Moment of inertia = $\frac{1}{12}\cdot b^3\cdot h$
l = Column height = 3,5 m
ρ = Dead load of concrete = 2400 kg/m^3
A = Column cross section = b*h [m^2]

If the material properties are known the lowest natural frequency for a beam is:

$$f = \frac{\pi}{2\cdot l^2}\cdot\sqrt{\frac{E\cdot I}{\rho\cdot A_l}} = \frac{\pi}{2\cdot l^2}\cdot\sqrt{\frac{E\cdot\frac{1}{12}\cdot b^3\cdot h}{\rho\cdot b\cdot h}} = \frac{\pi}{2\cdot l^2}\cdot\sqrt{\frac{E\cdot b^2}{12\cdot\rho}}$$

$$= \frac{\pi}{2\cdot 3,5^2}\cdot\sqrt{\frac{31\cdot 10^9\cdot 0.3^2}{12\cdot 2400}} = 39,91\,\text{s}^{-1}$$

$$T = \frac{1}{f} = \frac{1}{39,91} = 0,025\,\text{s}$$

$$\omega = f\cdot 2\pi = 39,91\cdot 2\pi = 250,76\,\text{s}^{-1}$$

Natural frequency walls

For non perpendicular walls:

$$f = \frac{\pi}{2}\cdot\left(\frac{1}{a^2}+\frac{1}{b^2}\right)\cdot\sqrt{\frac{E\cdot I}{\rho\cdot d\cdot\left(1-v^2\right)}} = \frac{\pi}{2}\cdot\left(\frac{1}{b^2}+\frac{1}{l^2}\right)\cdot\sqrt{\frac{E\cdot\frac{1}{12}\cdot b\cdot d^3}{\rho\cdot d\cdot\left(1-v^2\right)}}$$

$$= \frac{\pi}{2}\cdot\left(\frac{1}{b^2}+\frac{1}{l^2}\right)\cdot\sqrt{\frac{E\cdot\frac{1}{12}\cdot b\cdot d^2}{\rho\cdot\left(1-v^2\right)}} = \frac{\pi}{2}\cdot\left(\frac{1}{5^2}+\frac{1}{3,5^2}\right)\cdot\sqrt{\frac{31\cdot 10^9\cdot\frac{1}{12}\cdot 0,3\cdot 5^2}{2400\cdot\left(1-0,2^2\right)}} = 554,05\ \text{s}^{-1}$$

$$T = \frac{1}{f} = \frac{1}{554,05} = 0,002 \text{ s}$$

$$\omega = f \cdot 2\pi = 554,05 \cdot 2\pi = 3481 \text{ s}^{-1}$$

For perpendicular walls:

$$f = \frac{\pi}{2} \cdot \left(\frac{1}{a^2} + \frac{1}{b^2} \right) \cdot \sqrt{\frac{E \cdot I}{\rho \cdot d \cdot \left(1 - v^2\right)}} = \frac{\pi}{2} \cdot \left(\frac{1}{b^2} + \frac{1}{l^2} \right) \cdot \sqrt{\frac{E \cdot \frac{1}{12} \cdot d \cdot b^3}{\rho \cdot d \cdot \left(1 - v^2\right)}}$$

$$= \frac{\pi}{2} \cdot \left(\frac{1}{b^2} + \frac{1}{l^2} \right) \cdot \sqrt{\frac{E \cdot \frac{1}{12} \cdot b^3}{\rho \cdot \left(1 - v^2\right)}} = \frac{\pi}{2} \cdot \left(\frac{1}{5^2} + \frac{1}{3,5^2} \right) \cdot \sqrt{\frac{31 \cdot 10^9 \cdot \frac{1}{12} \cdot 0,3^3}{2400 \cdot \left(1 - 0,2^2\right)}} = 33,24 \text{ s}^{-1}$$

$$T = \frac{1}{f} = \frac{1}{33,24} = 0,030 \text{ s}$$

$$\omega = f \cdot 2\pi = 33,24 \cdot 2\pi = 208,85 \text{ s}^{-1}$$

Conclusion natural frequency

	Natural period T [s]	Natural frequency f [s^{-1}]	Angular rotation ω [s^{-1}]
Global building	0,70	1,43	8,98
Glass	0,045	22,23	139,68
Core	0,70	1,43	8,98
Floor	0,090	11,16	70,12
Column	0,025	39,91	250,76
Non perp. walls	0,002	554,05	3481
Perp. walls	0,030	33,24	208,85

Table 5-2: Natural frequency of a concrete building of 35 m high and its elements

Ductility elements

Ductility glass

Because glass is very brittle the ductility is equal to 1.

Ductility core

The ductility of the core is assumed to be comparable to the ductility of the whole building. Du =4.

Ductility Floor

The ductility of the floor is assumed to be comparable to the ductility of the whole building. Du =4.

Ductility column

The tensile reinforcement percentage is 1,5 %.

$$Du_{column} = \frac{0,1}{\omega_0 - \omega_0'}$$

Where:
ω_0 = Percentage tensile reinforcement [%]
ω'_0 = Percentage compression reinforcement [%]

$$Du_{column} = \frac{0,1}{0,015 - 0} = 7$$

Ductility wall

The ductility of the wall is considered to be comparable to that of the floor.

Conclusion ductility

	Ductility
Global building	4
Glass	1
Core	4
Floor	4
Column	7
Wall	4

Table 5-3: Natural frequency of a concrete building of 35 m high

Note 1:
Much uncertainty about the accuracy of the value of the ductility.

5.4 Damage case

In this chapter the damage to the building is calculated.

5.4.1 Chance of collapse

The scaled impulse and scaled pressure must be substituted.

$$\bar{i} = \frac{I \cdot \omega}{P_{st}} = \frac{1{,}98 \cdot 8{,}98}{5{,}1} = 3{,}49$$

$$\overline{P} = \frac{P_r}{P_{st}} = \frac{101}{5{,}1} = 19{,}80$$

With probit functions the chance of damage can be determined analytically. For buildings higher than four floors the probit on (partial) collapse as a result of a shock wave is given by the following formulae.

$$V=\left(\frac{0{,}9}{\overline{P}}\right)^{1{,}4}+\left(\frac{3}{\overline{i}}\right)^{2{,}7}$$

$$\Pr=5-2{,}92\cdot\ln V=5-2{,}92\cdot\ln\left(\left(\frac{0{,}9}{\overline{P}}\right)^{1{,}4}+\left(\frac{3}{\overline{i}}\right)^{2{,}7}\right)$$

$$=5-2{,}92\cdot\ln\left(\left(\frac{0{,}9}{19{,}80}\right)^{1{,}4}+\left(\frac{3}{3{,}49}\right)^{2{,}7}\right)=6{,}14$$

The chance of collapse can be read out from the table below and is 87 %.

%	0	1	2	3	4	5	6	7	8	9
0	--	2.67	2.95	3.12	3.25	3.36	3.45	3.52	3.59	3.66
10	3.72	3.77	3.82	3.897	3.92	3.96	4.01	4.05	4.08	4.12
20	4.16	4.19	4.23	4.26	4.29	4.33	4.36	4.39	4.42	4.45
30	4.48	4.50	4.53	4.56	4.59	4.61	4.64	4.67	4.69	4.72
40	4.75	4.77	4.80	4.82	4.85	4.87	4.90	4.92	4.95	4.97
50	5.00	5.03	5.05	5.08	5.10	5.13	5.15	5.18	5.20	5.23
60	5.25	5.28	5.31	5.33	5.36	5.39	5.41	5.44	5.47	5.50.
70	5.52	5.55	5.58	5.61	5.64	5.67	5.71	5.74	5.77	5.81
80	5.84	5.88	5.92	5.95	5.99	6.04	6.08	6.13	6.18	6.23
90	6.28	6.34	6.41	6.48	6.55	6.64	6.75	6.88	7.05	7.33
--	0.0	0.1	0.3	0.3	0.4	0.5	0.6	0.7	0.8	0.9
99	7.33	7.37	7.41	7.46	7.51	7.58	7.65	7.75	7.88	8.09

Table 5-4: Chance of collapse for probit

5.4.2 Equivalent static load

The equivalent static load is.

$$P_{eq.staticload}=I\cdot\omega=1{,}98\cdot 8{,}98=17{,}8$$

Which is larger than the static strength of the building P_{st} = 5,1 kPa. So the building will collapse.

5.4.3 Required ductility

The ductility that the building needs to take on the imposed impulse can be determined.

$$Du_{required}=\frac{1}{2}\left(\left(\frac{I\cdot\omega}{P_{st}}\right)^{2}+1\right)=\frac{1}{2}\left(\left(\frac{1{,}98\cdot 8{,}98}{5{,}1}\right)^{2}+1\right)=7$$

The building will collapse because the required ductility 7 is larger than the determined ductility of the building, 4.

5.4.4 Damage to elements

Chance of collapse

The probit function is only for buildings with more than four floors. It cannot be applied to separate elements. Except for glass for which a separate equation is given. This equation applies for buildings build after 1975.

$$\begin{aligned} &\Pr = -16,58 + 2,53 \cdot \ln Ps \\ &= -16,58 + 2,53 \cdot \ln 52000 = 10,89 \end{aligned}$$

The chance of failure of the glass is more than 99,9 %.

Equivalent static load

There is no damage if the calculated static strength is larger than the equivalent static load.

$$\begin{aligned} &P_{st} \geq P_{eq.staticload} \\ &P_{eq.staticload} = I \cdot \omega \\ &I = 1{,}98 \text{ kPa} \cdot \text{s} \end{aligned}$$

Element	Static strength (kPa)	ω (s^{-1})	$P_{eq.staticload}$ (kPa)
Global building	5,1	8,98	17,8
Glass	10,5	139,68	276,6
Core	14,4	8,98	17,8
Floor	1296	70,12	138,8
Column	77,7	250,77	496,5
Perp. walls	77,7	208,85	413,52
Non perp. walls	21572	3481	6892,38

Table 5-5: Static strength of the elements of the model building

Only the calculated equivalent static load of the floor and the non perpendicular walls is lower than the corresponding static strength. According to this method the rest of the elements will fail.

The impulse decreases with distance. Different points on the façade have different distances to the source so the impulse on the building also changes. This means that also the equivalent static load will change with distance.

		Width				
		0	5	10	15	20
Height	0	275	265	239	209	179
	5	265	256	232	204	176
	10	239	232	214	191	167
	15	209	204	191	173	155
	20	179	176	167	155	141
	25	155	152	146	138	127
	30	134	133	129	122	115
	35	118	117	114	109	104

Table 5-6: Equivalent static load on glass [kPa]

		Width				
		0	5	10	15	20
Height	0	494	476	430	374	322
	5	476	459	417	366	316
	10	430	417	384	342	300
	15	374	366	342	311	278
	20	322	316	300	278	253
	25	278	274	263	247	229
	30	241	239	231	220	206
	35	212	210	204	196	186

Table 5-7: Equivalent static load on column [kPa]

		Width				
		0	5	10	15	20
Height	0	138	133	120	105	90
	5	133	128	117	102	88
	10	120	117	107	96	84
	15	105	102	96	87	78
	20	90	88	84	78	71
	25	78	77	73	69	64
	30	67	67	65	61	58
	35	59	59	57	55	52

Table 5-8: Equivalent static load on floor [kPa]

The values of the equivalent static load on the glass and the column all exceed their maximum static strength, so all windows and columns will fail. There is only one core so the distribution of the impulse on the façade is not of importance. All the values of the equivalent static strength of the floor are smaller than the static strength of the floor; the floor will not fail.

Required ductility

If the ductility of the element is lower than the required ductility the structure will fail.

$$Du \geq Du_{required}$$

Method 1

$$Du_{required} = \frac{1}{2}\left(\left(\frac{I \cdot \omega}{P_{st}}\right)^2 + 1\right)$$

The minimal required ductility is at the far top end of the building of 35 m high and 20 wide. The maximal required ductility is at ground level perpendicular to the blast wave. I_{max} = 1,98 kPa*s and I_{min} = 0,74 kPa*s.

	P_{st}	ω	Du	$Du_{required,\min}$	$Du_{required,\max}$
Glass	10,5	139,68	1	49	347
Core	14,4	8,98	4	1	1
Floor	1296	70,12	4	1	1
Column	77,7	250,77	7	3	21
Perp. wall	77,7	208,82	7	2	15
Non perp. wall	21572	3481	4	1	1

Table 5-9: Required ductility elements method 1

Based on this required ductility all windows will fail but the core and floors will be intact. The columns at the foot of the building will fail but some columns at the upper floor are not destroyed in the blast.

Method 2

The positive phase duration is

$$t_p = 0,040 \text{ s}$$

For this method an other DLF is used.

$$DLF = \frac{P_{st}}{P_r}$$

The peak reflected overpressure has a non-uniform distribution on the façade. Which also means that the DLF changes.

		Width				
		0	5	10	15	20
Height	0	101	97	87	74	63
	5	97	93	84	72	62
	10	87	84	76	67	58
	15	74	72	67	61	54
	20	63	62	58	54	48
	25	54	53	50	47	43
	30	46	45	44	42	39
	35	40	40	38	37	35

Table 5-10: Peak reflected overpressure (kPa)

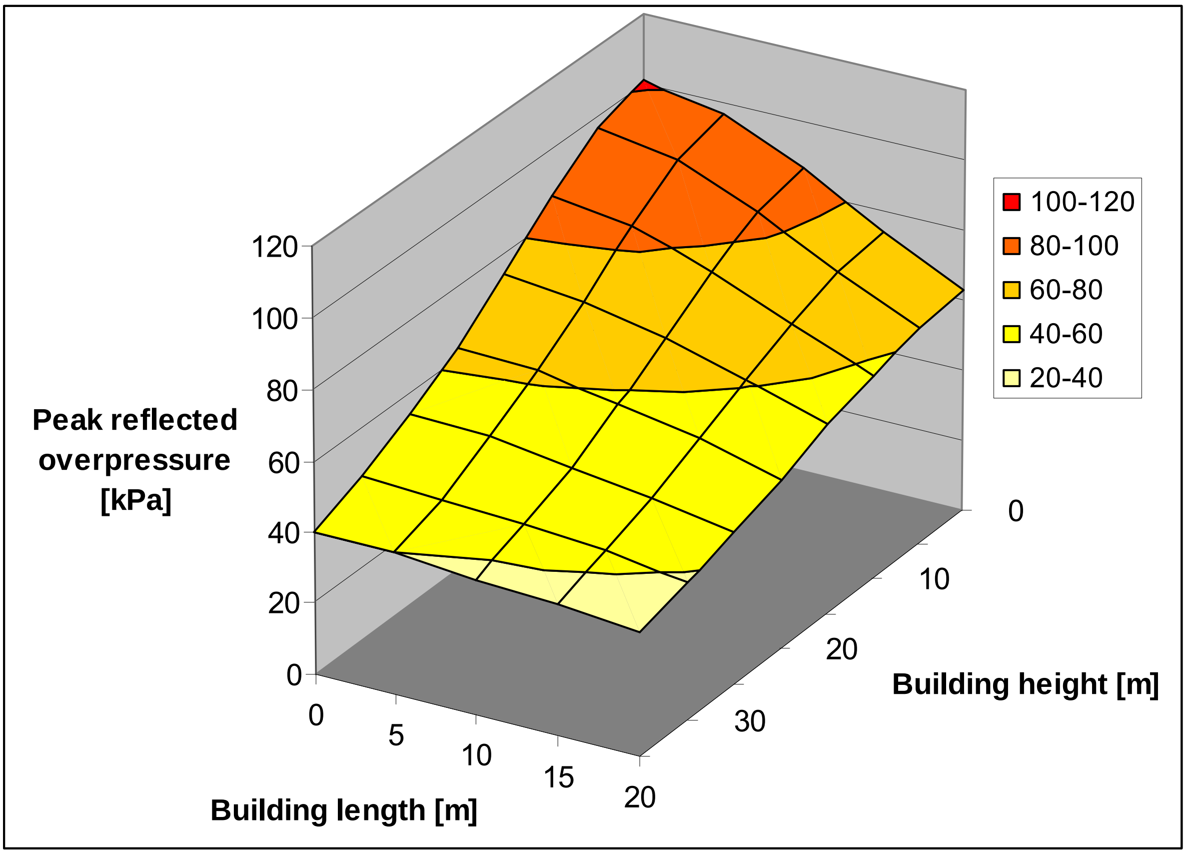


Figure 5-12: Distribution peak reflected overpressure

The minimal DLF can be found at a width and height of 0 meters. The maximum DLF is at 20 wide and 35 high at the far end corner of the building.

	T	P_{st}	DLF min	DLF max	$\frac{t_p}{T}$	$Du_{required,\min}$	$Du_{required,\max}$
Glass	0,045	10,5	0,10	0,30	0,9	40	-
Core	0,700	14,4	0,14	0,41	0,1	0,7	4
Floor	0,090	1296	12,83	37,25	0,4	-	-
Column	0,025	77,7	0,77	2,22	1,6	-	5,5
Perp. wall	0,030	77,7	0,77	2,22	1,3	-	4
Non perp. wall	0,002	21572	213,61	616,34	20	-	-

Table 5-11: Required ductility elements method 2

This method is only valid when tp << T or roughly tp/T <0,5 see figure 4-22. So the values for the glass and the column and walls are not valid. The core and the floor will not fail

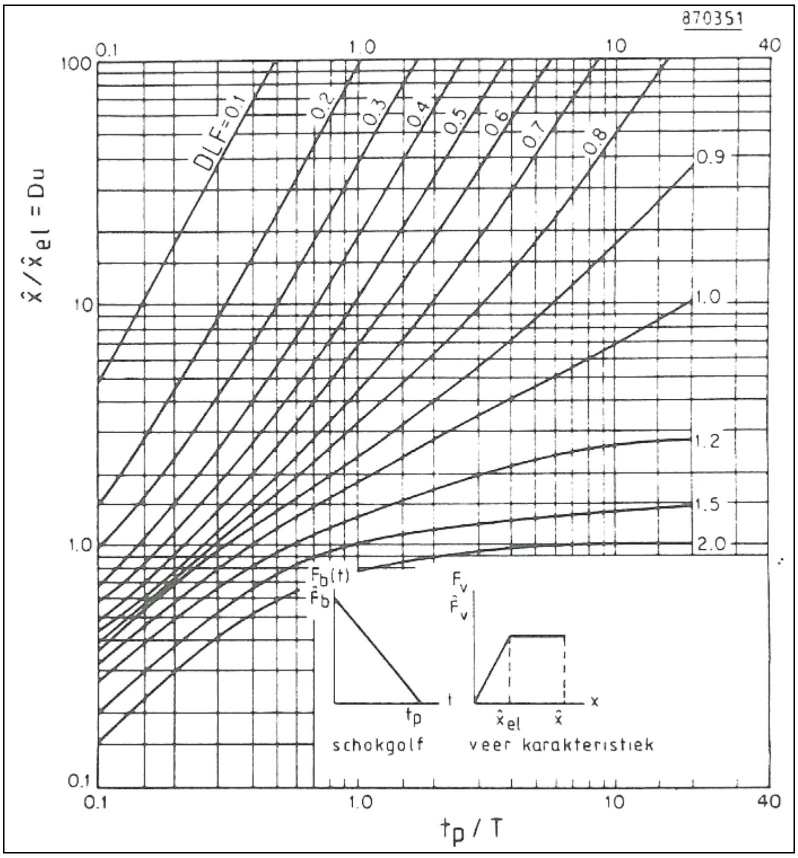


Figure 5-13: Response of an elastic-plastic one-mass-spring system to a shock wave [7].

Method 3

In the same way as in method 1 and 2 the minimal and maximum impulse and peak overpressure is used.

$$\bar{i} = \frac{i \cdot \omega}{P_{st}}$$

$$\bar{P} = \frac{P}{P_{st}}$$

	P_{st}	ω	$\bar{i}$ min	$\bar{i}$ max	$\bar{P}$ min	$\bar{P}$ max	$Du_{required,\min}$	$Du_{required,\max}$
Glass	10,5	139,68	10	26	3	10	>10	-
Core	14,4	8,98	1	1	2	7	1	1
Floor	1296	70,12	0	0	0	0	<1	<1
Column	77,7	250,77	2	6	0	1	<1	1<Du<5
Perp. wall	77,7	208,82	2	5	0	1	<1	1<Du<5
Non perp. wall	21572	3481	0	0	0	0	<1	<1

Table 5-12: Required ductility elements method 3

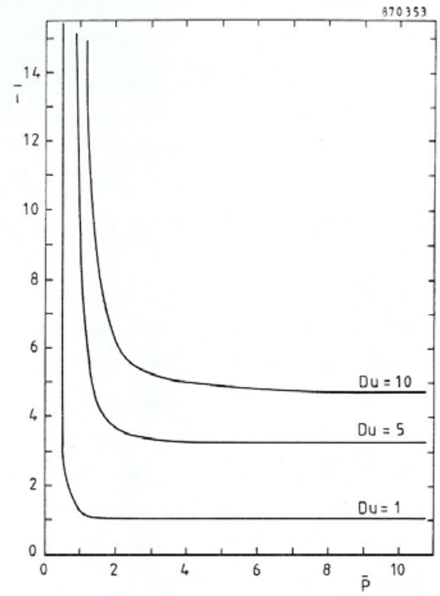


Figure: Scaled pressure impulse diagram

It can be concluded that the glass will fail but that the core, the floors, the walls and the columns will be intact.

5.5 Sensitivity analysis

By changing the parameters in the used methods slightly the results will be different. For several parameters these results are calculated and presented in the following tables. For every method used to define the load, the response aspects, the mechanical properties and the damage a sensitivity analysis is performed. The level in which the parameter is changed is chosen based on feasibility and reason. If a slight change of the parameter results in a large change of the outcome of the method this means that the method is very sensitive for that specific parameter. These sensitive parameters that have a large influence on the blast resistance of the building are of interest to increase this resistance with minimal effort. By changing these parameters a reduction of the damage can be achieved. When a parameter turns out to be sensitive it is recommended to do further study to define a finer method to determine the parameter.

5.5.1 Sensitivity analysis load and mechanical properties model building

The sensitivity analysis of each method is presented in a table. The first row of the table gives the parameters that determine the outcome of the method. In the second row the values of the parameters of the model building are given. In the third row the values of the parameters are changed in comparison with the values in the second row. The second column gives the values for the static strength and impulse of the model building. The changed values of the static strength and the impulse as a result of the changed parameter are given in the fourth to six row, in the third to seventh column. In the eighth column the values of the static strength and impulse are given if all the changed parameters are combined.

Parameter	Values Model building	Volume tank truck V [m^3]	Liquid density ρ [kg/m^3]	Distance R [m]	Atmospheric pressure P_0 [kPa]	Drag C_D [-]	rc [-]	Combo
Values model building	-	50	509	20	100	1,05	2,5	-
Changed value	-	20	435	40	95	0,8	2,2	-
P_s	52	36	44	22	49	52	52	14
P_r	101	67	85	40	96	101	92	25
I_s	1,97	1,00	1,58	0,85	1,87	1,97	1,79	0,38

Table 5-13: Sensitivity analysis load and response aspects model building

Clearly a larger distance is the best way to reduce the load on the building. Also smaller vessel volumes and lower liquid density can have a considerable reduction. A reduction of the atmospheric pressure, the drag and the reflection of the building do not result in a considerable smaller overpressure.

The reduction of the reflected overpressure as a result of a reduction of the rc results in a different value than calculated in the chapter about a round building vs. a rectangular building. This is because in that chapter also the width of the building is taken into account.

Because the positive phase duration is smaller than the t_s there will be no pressure relief wave and the building width B, drag coefficient C_D, dynamic wind load Q_D and the speed of the wave front U are of no influence to the value of the reflected peak overpressure.

Parameter	Values Model building	Safety factor steel β [-]	Safety factor wood β [-]	Height H [m]	Location	Combined
Values model building	-	2,7	2,7	35	Area II	-
Changed value	-	2,0	5,0	100	Area I	-
Static strength P_{st} [kPa]	5,1	3,8	9,4	6,4	5,9	13,3

Table 5-14: Sensitivity analysis static strength model building

The static strength of a concrete building is because of the safety factor larger than that of a steel building and smaller than that of a wooden building. Higher buildings result in higher design wind loads on the building and therefore making it also better resistant to a blast load. Also when using the wind load norm on buildings in area I the static strength of the building will be larger. Overall it can be stated that a larger horizontal design load results in a larger static strength of the building.

Parameter	Values model building	Building height [m]
Values model building	-	35
Changed value	-	100
Natural period T [s]	0,72	2
Natural frequency f [s^{-1}]	1,38	0,5
Angular rotation ω [s^{-1}]	8,67	3,14

Table 5-15: Sensitivity analysis natural period

The only parameter is the height of the building, which shows a large sensitivity. The building should be as high as possible.

A sensitivity analysis of the method to determine the ductility is not possible because the ductility is not determined with a method.

In conclusion it can be stated that the most sensitive parameters of the load and mechanical properties of the building are the distance to the explosion, the volume of the tank, the horizontal design load and the stiffness of the building (largely determined by the height of the building). When designing a building and keeping the blast resistance in mind these are the parameters that should be optimised.

5.5.2 Sensitivity analysis mechanical properties elements

Sensitivity analysis static strength elements

In the tables below a sensitivity analyses is given of the methods that determine the static strength of the elements.

Parameter	Values Model building	Smallest dimension a [m]	Largest dimension b [m]	Thickness d [m]	Modulus of Elasticity E [N/m^2]	Combo
Values model building	-	1	1,5	0,006	75*10^9	-
Changed value	-	0,8	1,3	0,008	73*10^9	-
Static strength P_{st} [kPa]	10,5	13,3	11,1	13,7	10,6	19,0

Table 5-16: Sensitivity analysis static strength glass

The dimensions of the glass determine the static strength of the window.

Parameter	Values Model building	Percentage rebar [%]	Width core [m]	Wall thickness core [m]	Length building [m]	Height building [m]	Combo
Values model building	-	1,5	5	0,3	20	35	-
Changed value	-	2,0	7	0,4	15	30	-
Static strength P_{st} [kPa]	14,4	19,2	28,2	19,2	19,2	19,6	90,9

Table 5-17: Sensitivity analysis static strength core

The static strength of the core depends mostly on the width of the core. The other parameters also have considerable sensitivity. If the width of the core is 20 m equal to the width of the building (load bearing façade) the static strength becomes 230,1 kPa.

Parameter	Values Model building	Thickness floor h [m]	Width of building B [m]	Length of building L [m]	Story height S [kg/m^3]	Combo
Values model building	-	0,3	20	20	3,5	
Changed value	-	0,4	25	15	3	
Static strength P_{st} [kPa]	1296	1728	1620	1728	1512	3360

Table 5-18: Sensitivity analysis static strength floor

The floor thickness and the length of the building determine largely the static strength of the floor, which is very large compared to the other elements.

Parameter	Values model building	Percentage rebar ω_0 [%]	Width column b [m]	Length column l [m]	Combo
Values model building	-	1,5	0,3	3,5	
Changed value	-	2,0	0,4	3	
Static strength P_{st} [kPa]	77,7	103,5	138,1	105,7	250,6

Table 5-19: Sensitivity analysis static strength column and perpendicular walls

Parameter	Values model building	Percentage rebar ω_0 [%]	Width wall [m]	Height wall [m]	Combo
Values model building	-	1,5	5	3,5	
Changed value	-	2,0	10	3	
Static strength P_{st} [kPa]	21572	28763	86290	29363	156600

Table 5-20: Sensitivity analysis static strength non perpendicular walls

Sensitivity analysis natural frequency elements

Parameter	Values Model building	Smallest dimension a [m]	Largest dimension b [m]	Thickness d [m]	Modulus of Elasticity E [N/m^2]	Specific mass glass ρ [kg/m^3]	Combo
Values model building	-	1	1,5	0,006	$75*10^9$	2500	-
Changed value	-	0,8	1,3	0,008	$77*10^9$	2300	-
Natural frequency [s^{-1}]	22,23	19,20	20,72	14,82	21,93	21,80	11,31
Natural period [ms]	45	52	48	67	46	46	88
Angular rotation [s^{-1}]	139,68	120,64	130,16	93,12	137,81	136,97	71,08

Table 5-21: Sensitivity analysis natural frequency glass

Again the dimensions of the glass have the largest influence, especially the thickness.

Parameter	Values Model building	Width building B [m]	Length building L [m]	Modulus of Elasticity E [N/m^2]	Thickness floor h [m]	Specific mass glass ρ [kg/m^3]	Combo
Values model building	-	20	20	$31*10^9$	0,3	2400	-
Changed value	-	25	25	$29*10^9$	0,25	2600	-
Natural frequency [s^{-1}]	11,16	10,23	9,15	10,79	9,30	10,72	6,18
Natural period [ms]	90	98	109	93	108	93	162
Angular rotation ω [s^{-1}]	70,11	64,27	57,49	67,81	58,42	67,36	38,85

Table 5-22: Sensitivity analysis natural frequency floor

To reduce the angular rotation of the floor and the column the width and length should be maximized and the thickness should be minimized.

Parameter	Values Model building	Width column b [m]	Column length l [m]	Modulus of Elasticity E [N/m^2]	Specific mass glass ρ [kg/m^3]	Combo
Values model building	-	0,3	3,5	$31*10^9$	2400	-
Changed value	-	0,25	4,0	$29*10^9$	2600	-
Natural frequency [s^{-1}]	39,91	33,26	30,56	38,60	38,34	23,66
Natural period [ms]	25	30	33	26	26	42
Angular rotation [s^{-1}]	250,77	208,97	191,99	242,54	240,93	148,68

Table 5-23: Sensitivity analysis natural frequency column

Parameter	Values Model building	Wall length [m]	Wall height [m]	Modulus of Elasticity E [N/m^2]	Thickness floor h [m]	Specific mass glass ρ [kg/m^3]	Combo
Values model building	-	5	3,5	$31*10^9$	0,3	2400	-
Changed value	-	10	4,0	$29*10^9$	0,25	2600	-
Natural frequency [s^{-1}]	33,24	25,04	28,01	32,15	25,29	31,94	14,01
Natural period [ms]	30	40	36	31	40	31	71
Angular rotation ω [s^{-1}]	208,87	157,36	176,02	202,02	158,89	200,68	88,01

Table 5-24: Sensitivity analysis natural frequency perpendicular wall

Parameter	Values Model building	Width building B [m]	Length building L [m]	Modulus of Elasticity E [N/m^2]	Thickness floor h [m]	Specific mass glass ρ [kg/m^3]	Combo
Values model building	-	5	3,5	$31*10^9$	0,3	2400	-
Changed value	-	10	4,0	$29*10^9$	0,25	2600	-
Natural frequency [s^{-1}]	554,05	834,79	466,90	535,88	505,78	532,31	1780,94
Natural period [ms]	2	1	2	2	2	2	1
Angular rotation ω [s^{-1}]	3481,20	5245,17	2933,62	3367,04	3177,89	3344,63	11189,96

Table 5-25: Sensitivity analysis natural frequency non perpendicular wall

Sensitivity analysis ductility elements

A sensitivity analysis of the walls, the core and the glass is not possible because there is no method.

Parameter	Values Model building	Percentage tensile reinforcement ω_0 [%]	Percentage compression reinforcement ω_0 [%}	Combo
Values model building	-	1,5	0	-
Changed value	-	1,0	0,5	-
Ductility Du [-]	7	10	10	20

Table 5-26: Sensitivity analysis ductility beam and column

The ductility of the column is very sensitive for the percentage of reinforcement.

5.5.3 Sensitivity analysis damage to model building

Sensitivity analysis chance of collapse

Does it make any sense to chance the parameters to reduce the reflected overpressure and increase the static strength or will the methods that determine the damage always result in collapse, making the efforts to increase the static strength useless?

In the following table the chance of collapse is given of the building when certain parameters are changed. On the left are the parameters that determine the load, the values are taken from Table 5-13. On top are the parameters that determine the static strength, these are taken from Table 5-14. The angular rotation is unchanged. This results in the chance of collapse. It can be read from the table that with certain parameters the chance of collapse is smaller than 1. It can be concluded that according to this damage method it is possible to make the building able to withstand the blast.

	Concrete	Steel	Wood	Building heigth	Location	Combo
P_{st}	5,1	3,8	9,4	6,4	5,9	13,3
ω	8,98	8,98	8,98	3,14	8,98	3,14

	P_r	I	Probit					
Values model building	101	1,97	6,09	8,38	1,30	-3,93	4,95	-9,70
Volume tank truck	67	1,00	0,78	3,10	-4,03	-9,28	-0,36	-15,04
Liquid density	85	1,58	4,37	6,67	-0,43	-5,67	3,22	-11,44
Distance	40	0,85	-0,50	1,81	-5,31	-10,56	-1,65	-16,32
Atmospheric pressure	96	1,87	5,68	7,98	0,89	-4,34	4,54	-10,11
rc	92	1,79	5,34	7,64	0,55	-4,69	4,20	-10,45
Combo	25	0,38	-6,83	-4,52	-11,65	-16,90	-7,98	-22,67

Chance of collapse [%]					
1<x<99,9	99,9	<1	<1	1<x<99,9	<1
<1	1<x<99,9	<1	<1	<1	<1
1<x<99,9	1<x<99,9	<1	<1	1<x<99,9	<1
<1	<1	<1	<1	<1	<1
1<x<99,9	1<x<99,9	<1	<1	1<x<99,9	<1
1<x<99,9	1<x<99,9	<1	<1	1<x<99,9	<1
<1	<1	<1	<1	<1	<1

Table 5-27: Sensitivity analysis chance of collapse of model building.

It can be read from the table above that it is possible to reduce the chance of collapse (even to zero) by changing the load parameters and static strength parameters. The angular rotation is changed for when the building is higher.

If all parameters are the unchanged values of the model building, except the distance of the building from the source, the distance needed where the building will not collapse is 29 m. This distance is called the collapse radius.

When the peak reflected overpressure and the chance of collapse in table 5-27 are compared with the tables in paragraph 4.4.2. it gives a validation that the results are realistic.

Sensitivity analysis equivalent static load

In the same way as for the chance of collapse the equivalent static load can be determined for the different parameters used in the sensitivity analysis. The result is presented here.

	Concrete	Steel	Wood	Building heigth	Location	Combo
P_{st}	5,1	3,8	9,4	6,4	5,9	13,3
ω	8,98	8,98	8,98	3,14	8,98	3,14

	I	Equivalent static load [kPa]					
Values model building	1,97	17,7	17,7	17,7	6,2	17,7	6,2
Volume tank truck	1,00	9,0	9,0	9,0	3,1	9,0	3,1
Liquid density	1,58	14,2	14,2	14,2	5,0	14,2	5,0
Distance	0,85	7,6	7,6	7,6	2,7	7,6	2,7
Atmospheric pressure	1,87	16,8	16,8	16,8	5,9	16,8	5,9
rc	1,79	16,1	16,1	16,1	5,6	16,1	5,6
Combo	0,38	3,4	3,4	3,4	1,2	3,4	1,2

Collapse					
Yes	Yes	Yes	No	Yes	No
Yes	Yes	No	No	Yes	No
Yes	Yes	Yes	No	Yes	No
Yes	Yes	No	No	Yes	No
Yes	Yes	Yes	No	Yes	No
Yes	Yes	Yes	No	Yes	No
No	No	No	No	No	No

Table 5-28: Sensitivity analysis equivalent static load model building.

If the value of the equivalent static load is larger than the static strength the model is supposed to collapse. At a distance of 58 m the model building will experience no damage when all other parameters are kept the same as the model building.

Sensitivity analysis required ductility

Again by changing the parameters the required ductility can be changed. As shown in the table below the maximum ductility of 10 is well in reach, even a ductility of 1.

	Concrete	Steel	Wood	Building heigth	Location	Combo
P_{st}	5,1	3,8	9,4	6,4	5,9	13,3
ω	8,98	8,98	8,98	3,14	8,98	3,14

	I	Required ductility					
Values model building	1,97	7	11	2	1	5	1
Volume tank truck	1,00	2	3	1	1	2	1
Liquid density	1,58	4	7	2	1	3	1
Distance	0,85	2	3	1	1	1	1
Atmospheric pressure	1,87	6	10	2	1	5	1
rc	1,79	5	9	2	1	4	1
Combo	0,38	1	1	1	1	1	1

Collapse					
Maybe	Yes	No	No	Maybe	No
No	No	No	No	No	No
Maybe	Maybe	No	No	No	No
No	No	No	No	No	No
Maybe	Yes	No	No	Maybe	No
Maybe	Maybe	No	No	Maybe	No
No	No	No	No	No	No

Table 5-29: Sensitivity analysis required ductility of model building.

The required ductility can be reduced to 1 very easy. According to the ductility the building is consider not to collapse. The collapse radius, the distance at which the required ductility is reached without changing other parameters, is 25 m.

5.5.4 Sensitivity analysis damage to elements

In chapter 5.4.4. the damage of the elements of the model building was determined. In this chapter it is studied what the sensitivity of these methods are.

Chance of collapse

The change of collapse of the model building was

$$\begin{aligned}\Pr &= -16,58 + 2,53 \cdot \ln Ps \\ &= -16,58 + 2,53 \cdot \ln 52000 = 10,89\end{aligned}$$

Which resulted in a chance of collapse of 99,9 %. By changing the parameters of the glass the static strength can be increased and by changing the parameters of the load the side on overpressure can be reduced.

		No changes	Width glass	Length glass	Thickness glass	E modulus	Combo
P_{st}		10,5	13,3	11,1	13,7	10,6	19,0
	P_s	Probit					
Values model building	52	10,89	10,89	10,89	10,89	10,89	10,89
Volume tank truck	36	9,96	9,96	9,96	9,96	9,96	9,96
Liquid density	44	10,47	10,47	10,47	10,47	10,47	10,47
Distance	22	8,72	8,72	8,72	8,72	8,72	8,72
Atmospheric pressure	49	10,74	10,74	10,74	10,74	10,74	10,74
Combined	14	7,57	7,57	7,57	7,57	7,57	7,57
		Chance of collapse [%]					
		99,9	99,9	99,9	99,9	99,9	99,9
		99,9	99,9	99,9	99,9	99,9	99,9
		99,9	99,9	99,9	99,9	99,9	99,9
		99,9	99,9	99,9	99,9	99,9	99,9
		99,9	99,9	99,9	99,9	99,9	99,9
		1<x<99,9	1<x<99,9	1<x<99,9	1<x<99,9	1<x<99,9	1<x<99,9

Table 5-30: Sensitivity analyses chance of collapse glass

It is shown in this table that the optimization of the parameters does not lead to a window that will not fail. The chance of collapse in most cases is still 99,9 %. It is calculated that without changing any parameters except the distance from the explosion, this distance should be 236 m in order to prevent the windows from failing.

Equivalent static load

Chapter 5.4.4. resulted in the following table.

Element	Static strength (kPa)	ω (s^{-1})	$P_{eq.staticload}$ (kPa)
Global building	5,1	8,98	17,8
Glass	10,5	139,68	276,6
Core	14,4	8,98	17,8
Floor	1296	70,12	138,8
Column	77,7	250,77	496,5
Perp. walls	77,7	208,85	413,52
Non perp. walls	21572	3481	6892,38

Table 5-31: Static strength of the elements of the model building

Only the calculated equivalent static load of the floor and the non perpendicular wall is lower than the corresponding static strength. According to this method the rest of the elements will fail.

By changing the parameters of the static strength, the impulse and the angular rotation the equivalent static load can be reduced. A sensitivity analyses is performed for the glass, the core and the column and the perpendicular walls.

	I	No changes	Width glass	Length glass	Thickness glass	Modulus of elasticity	Specific weight	Combo
P_{st}		10,5	13,3	11,1	13,7	10,6	10,5	19,0
ω		139,68	120,6	130,16	93,12	137,81	136,97	71,08
		Equivalent static load [kPa]						
Values model building	1,97	275,2	237,7	256,4	183,4	271,5	269,8	140,0
Volume tank truck	1,00	139,7	120,6	130,2	93,1	137,8	137,0	71,1
Liquid density	1,58	220,7	190,6	205,7	147,1	217,7	216,4	112,3
Distance	0,85	118,7	102,5	110,6	79,2	117,1	116,4	60,4
Atmospheric pressure	1,87	261,2	225,6	243,4	174,1	257,7	256,1	132,9
rc	1,79	250,0	215,9	233,0	166,7	246,7	245,2	127,2
Combo	0,38	53,1	45,8	49,5	35,4	52,4	52,0	27,0
		Collapse						
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes

Table 5-32: Sensitivity analyses equivalent static load glass

If the calculated equivalent static load is larger than the static strength the glass will collapse. This table shows that changing the parameters did not help; the glass will fail. If all parameters are the unchanged model building parameters the building has to be at a distance of 363 m in order to prevent the windows from failing.

	I	No changes	Percentage rebar	Width core	Wall thickness core	Length building	Heigth building	Combo
P_{st}		14,4	19,2	28,2	19,2	19,2	19,6	90,9
ω		8,98	8,98	8,98	8,98	8,98	3,14	3,14
		Equivalent static load [kPa]						
Values model building	1,97	17,7	17,7	17,7	17,7	17,7	6,2	6,2
Volume tank truck	1,00	9,0	9,0	9,0	9,0	9,0	3,1	3,1
Liquid density	1,58	14,2	14,2	14,2	14,2	14,2	5,0	5,0
Distance	0,85	7,6	7,6	7,6	7,6	7,6	2,7	2,7
Atmospheric pressure	1,87	16,8	16,8	16,8	16,8	16,8	5,9	5,9
rc	1,79	16,1	16,1	16,1	16,1	16,1	5,6	5,6
Combo	0,38	3,4	3,4	3,4	3,4	3,4	1,2	1,2
		Collapse						
		Yes	No	No	No	No	No	No
		No	No	No	No	No	No	No
		No	No	No	No	No	No	No
		No	No	No	No	No	No	No
		Yes	No	No	No	No	No	No
		Yes	No	No	No	No	No	No
		No	No	No	No	No	No	No

Table 5-33: Sensitivity analyses equivalent static load core

The sensitivity analyses of the equivalent static load of the core shows that the unchanged model building will collapse but that in most other cases this method concludes that the building will not collapse. The model building will collapse when it is in a range of 24 m from the explosion.

	I	No changes	Width column	Length column	Percentage rebar	E modulus	Specific mass	Combo
P_{st}		77,7	138,1	105,7	103,5	77,7	77,7	250,6
ω		250,77	208,97	191,99	250,77	242,54	240,93	148,68
		Equivalent static load [kPa]						
Values model building	1,97	494,0	411,7	378,2	494,0	477,8	474,6	292,9
Volume tank truck	1,00	250,8	209,0	192,0	250,8	242,5	240,9	148,7
Liquid density	1,58	396,2	330,2	303,3	396,2	383,2	380,7	234,9
Distance	0,85	213,2	177,6	163,2	213,2	206,2	204,8	126,4
Atmospheric pressure	1,87	468,9	390,8	359,0	468,9	453,5	450,5	278,0
rc	1,79	448,9	374,1	343,7	448,9	434,1	431,3	266,1
Combo	0,38	95,3	79,4	73,0	95,3	92,2	91,6	56,5
		Collapse						
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	No
		Yes	Yes	Yes	Yes	Yes	Yes	No
		Yes	Yes	Yes	Yes	Yes	Yes	No
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	No	No	No	Yes	Yes	No

Table 5-34: Sensitivity analyses equivalent static load column

If no parameters are changed all columns in a radius of 100 m will fail. By changing the parameters the equivalent static strength can be reduced to a value lower than the static strength of the columns.

	I	No changes	Wall width	Wall height	Percentage rebar	Wall length	E modulus	Specific mass	Combi
P_{st}		77,7	138,1	105,7	103,5	77,7	77,7	77,7	140,9
ω		208,87	321,58	259,49	208,87	157,36	202,02	200,68	178,12
		Equivalent static load [kPa]							
Values model building	1,97	411,5	633,5	511,2	411,5	310,0	398,0	395,3	350,9
Volume tank truck	1,00	208,9	321,6	259,5	208,9	157,4	202,0	200,7	178,1
Liquid density	1,58	330,0	508,1	410,0	330,0	248,6	319,2	317,1	281,4
Distance	0,85	177,5	273,3	220,6	177,5	133,8	171,7	170,6	151,4
Atmospheric pressure	1,87	390,6	601,4	485,3	390,6	294,3	377,8	375,3	333,1
rc	1,79	373,9	575,6	464,5	373,9	281,7	361,6	359,2	318,8
Combo	0,38	79,4	122,2	98,6	79,4	59,8	76,8	76,3	67,7
		Collapse							
		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	No	No	No	No	No	No	No

Table 5-35: Sensitivity analysis equivalent static load perpendicular walls

The collapse radius of the perpendicular walls is 83 m.

Required ductility

All three methods to determine the ductility of the elements resulted in a required ductility lower than the actual ductility for the core, the floor and the non perpendicular walls. No sensitivity analysis is necessary here because according to the ductility the core, the floor and the non perpendicular walls will not collapse. For the glass, the column and the perpendicular walls a sensitivity analysis is performed for method 1. Not for method 2 because that method is not applicable for the glass and the column. See also chapter 5.4.4. (also for the tables).

	I	No changes	Width glass	Length glass	Thickness glass	E modulus	Specific weight	Combo
P_{st}		10,5	13,3	11,1	13,7	10,6	10,5	19,0
ω		139,68	120,6	130,16	93,12	137,81	136,97	71,08
		Required ductility [kPa]						
Values model building	1,97	344	160	267	90	328	331	28
Volume tank truck	1,00	89	42	69	24	85	86	7
Liquid density	1,58	221	103	172	58	211	213	18
Distance	0,85	64	30	50	17	62	62	6
Atmospheric pressure	1,87	310	144	241	81	296	298	25
rc	1,79	284	132	221	75	271	273	23
Combo	0,38	13	6	10	4	13	13	2
		Collapse						
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes
		Yes	Yes	Yes	Yes	Yes	Yes	Yes

Table 5-36: Sensitivity analyses required ductility glass.

From these tables can be read that the changed parameters do not lead to a required ductility as low as the actual ductility of glass of Du=1. All windows will fail in a radius of 363 m.

	I	No changes	Percentage rebar	Width column	Length of column	E Modulus	Specific mass	Combo
P_{st}		77,7	103,5	138,1	105,7	77,7	77,7	250,6
ω		250,77	250,77	208,97	191,99	242,54	240,93	148,68
		Required ductility						
Values model building	1,97	21	12	5	7	19	19	1
Volume tank truck	1,00	6	3	2	2	5	5	1
Liquid density	1,58	14	8	3	5	13	13	1
Distance	0,85	4	3	1	2	4	4	1
Atmospheric pressure	1,87	19	11	5	6	18	17	1
rc	1,79	17	10	4	6	16	16	1
Combo	0,38	1	1	1	1	1	1	1
		Collapse						
		Yes	Yes	No	No	Yes	Yes	No
		No	No	No	No	No	No	No
		Yes	Yes	No	No	Yes	Yes	No
		No	No	No	No	No	No	No
		Yes	Yes	No	No	Yes	Yes	No
		Yes	Yes	No	No	Yes	Yes	No
		No	No	No	No	No	No	No

Table 5-37: Sensitivity analysis required ductility column.

In a radius of 32 m from the explosion the required ductility does not reach the actual ductility of the columns, so they will collapse.

	No changes	Percentage rebar	Wall width	Wall height	Wall length	E modulus	Specific mass	Combo
P_{st}	77,7	103,5	138,1	105,7	77,7	77,7	77,7	250,6
ω	208,87	208,87	321,58	259,49	157,36	202,02	200,68	88,01

	l	Required ductility							
Values mo	1,97	15	8	11	12	8	14	13	1
Volume tar	1,00	4	3	3	4	3	4	4	1
Liquid dens	1,58	10	6	7	8	6	9	9	1
Distance	0,85	3	2	2	3	2	3	3	1
Atmospher	1,87	13	8	10	11	8	12	12	1
rc	1,79	12	7	9	10	7	11	11	1
Combo	0,38	1	1	1	1	1	1	1	1

Collapse							
Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Yes	No	No	No	No	No	No	No
Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
No	No	No	No	No	No	No	No
Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
No	No	No	No	No	No	No	No

Table 5-38: Sensitivity analysis required ductility perpendicular walls.

The collapse radius of perpendicular walls according to this method is 36 m.

5.5.5 Conclusion sensitivity analyses damage

	Damage method		
	Chance of collapse	Equivalent static load	Required ductility
Global building	29 m	58 m	25 m
Glass	236 m	363 m	363 m
Core	-	24 m	12 m
Floor	-	<1 m	<1 m
Column	-	100 m	32 m
Perp. wall	-	83 m	36 m
Non perp. wall	-	9 m	5 m

Table 5-39: Theoretical collapse radius for different elements and damage methods

For the whole building and all elements the required ductility results in a lower collapse radius than with the method that determines the equivalent static load. This means that not the required ductility but the equivalent static load on the building is governing. There is however much uncertainty about the methods that determine the ductility of the elements.

When only the equivalent static load is governed the collapse radius of the five structural elements the core, the floor, the column, the perpendicular wall and the non perpendicular wall are respectively 24 m, <1 m, 100 m, 83 m and 9 m. The collapse radius of the column is governing. The static strength of the column should be as high or higher than the equivalent static load in order to make it withstand the blast.

The formulas for the static strength and the equivalent static load of the column are:

$$P_{st,column} = \frac{2 \cdot \omega_0 \cdot b^2 \cdot f_s \cdot \beta \cdot 0{,}9}{3 \cdot l^2}$$

$$P_{eq.staticload} = \frac{\pi^2}{l^2} \cdot \sqrt{\frac{E \cdot b^2}{12 \cdot \rho}}$$

The parameters that can reduce the collapse radius are for instance the width and length of the column and the percentage of tensile reinforcement. A contradiction occurs. In order to reduce the equivalent static load the angular rotation should be minimized. A larger column width and smaller column length result in a larger static strength but it also results in a larger angular rotation.

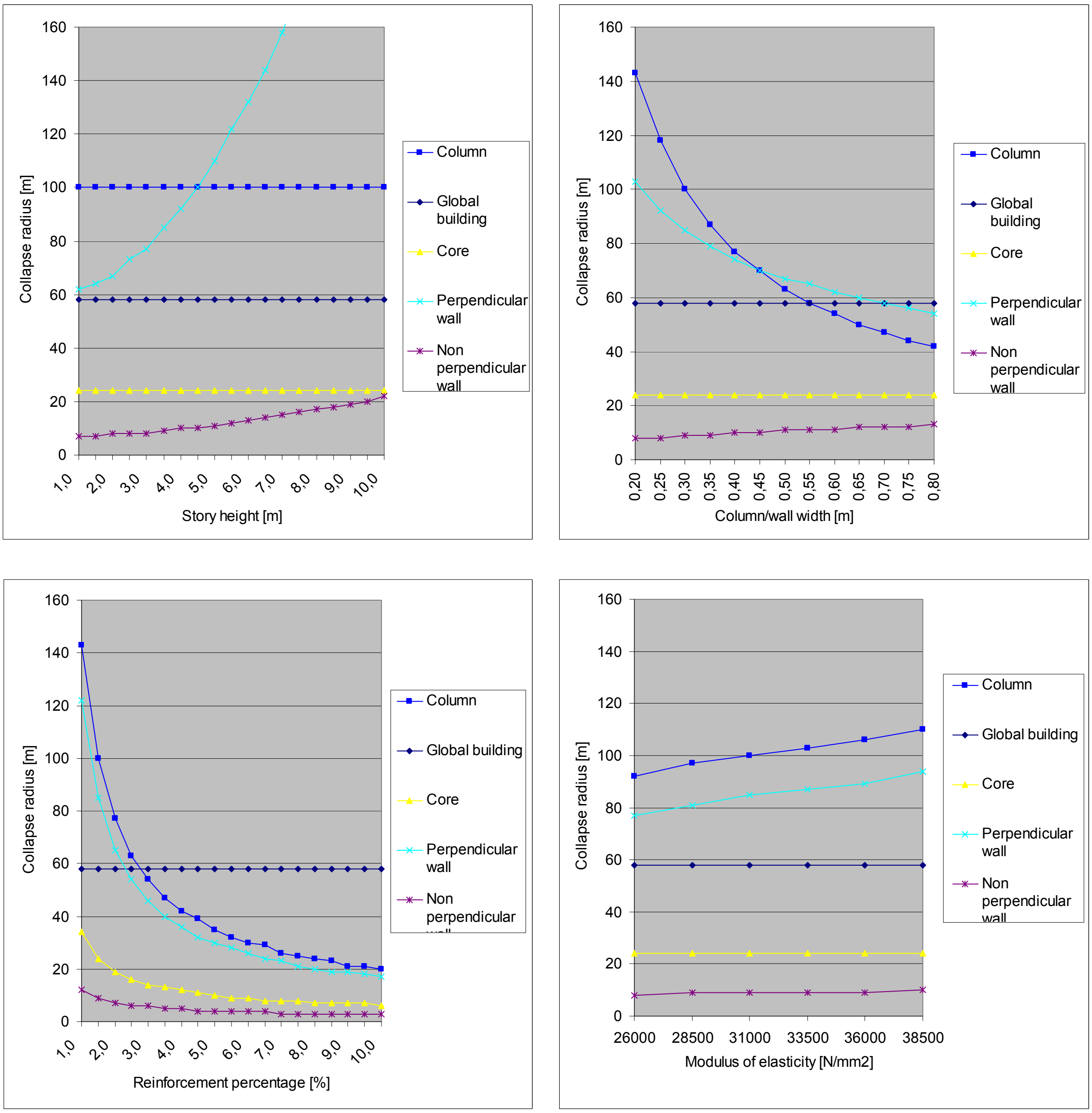


Figure 5-14: Graphs sensitivity analyses collapse radius column, global building, core, perpendicular wall and non perpendicular wall.

The graph shows that the core and the non perpendicular walls are never the governing structural element. Also can be seen that an increase or decrease of the story height does not help to decrease the collapse radius of the column, when the story height is more than 4,5 m it increases the collapse radii of the walls. An increase of the tensile reinforcement and the column/wall width do help to decrease the collapse radius, also a decrease of the modulus of elasticity helps.

When the column width is larger than 0,45 m it is not the governing structure element any more, as can be read from the graph. In order to reduce the collapse radius further not only the collapse radius of the column should be further reduced also the collapse radius of the perpendicular wall should be reduced. And when the width is even further increased to 0,70 m also the collapse radius of the global building has to be decreased.

In conclusion; the most important parameters to increase the blast-resistance or reduce the collapse radius are:

- Column/wall width
- Horizontal design load
- Building height

When no columns or perpendicular walls are used in the building structure the governing element is always the global building. It's collapse radius can be decreased by either increasing the design load which increases the static strength or increasing the height of the building to reduce the natural frequency. When the horizontal wind load in wind area I is used the following graphs applies.

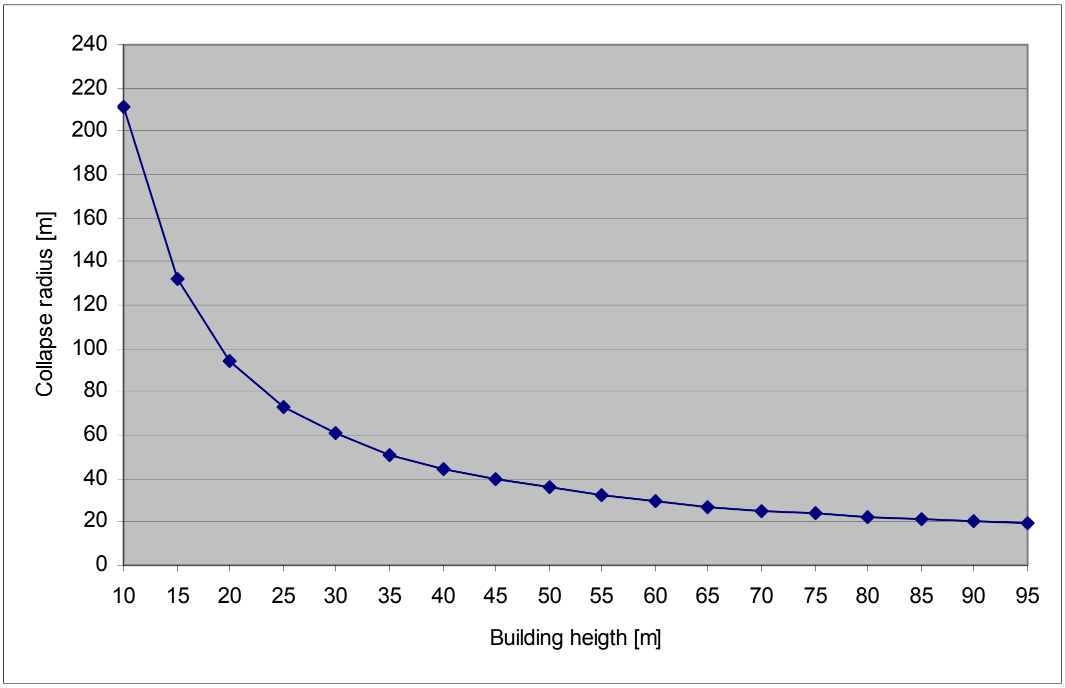


Figure 5-15: Collapse radius vs. Building height.

The wind load in area I should be multiplied with a factor 4,5 in order to make a 25 m high building at a distance of 20 meters from the source not to collapse. This explosion factor has been determined iteratively and differs for the height of the building and the distance from the source.

Building height [m]	Explosion factor [-]
5	33,1
10	13,9
15	8,5
20	6,0
25	4,5
30	3,7
35	3,0
40	2,6
45	2,2
50	2,0
55	1,8
60	1,6
65	1,4
70	1,3
75	1,2
80	1,1
85	1,1
90	1,0
95	1,0
100	1,0
105	1,0
110	1,0

Table 5-40: Explosion factor collapse radius 20 m.

When the governing wind load in wind area I in the code NEN6702 is multiplied with this explosion factor the building will not collapse if placed at a minimum distance of 20 m. This only applies for buildings without separate columns and beams but only shear walls in the direction of the load and floors. The graph below gives the explosion factor for several collapse radii and building heights.

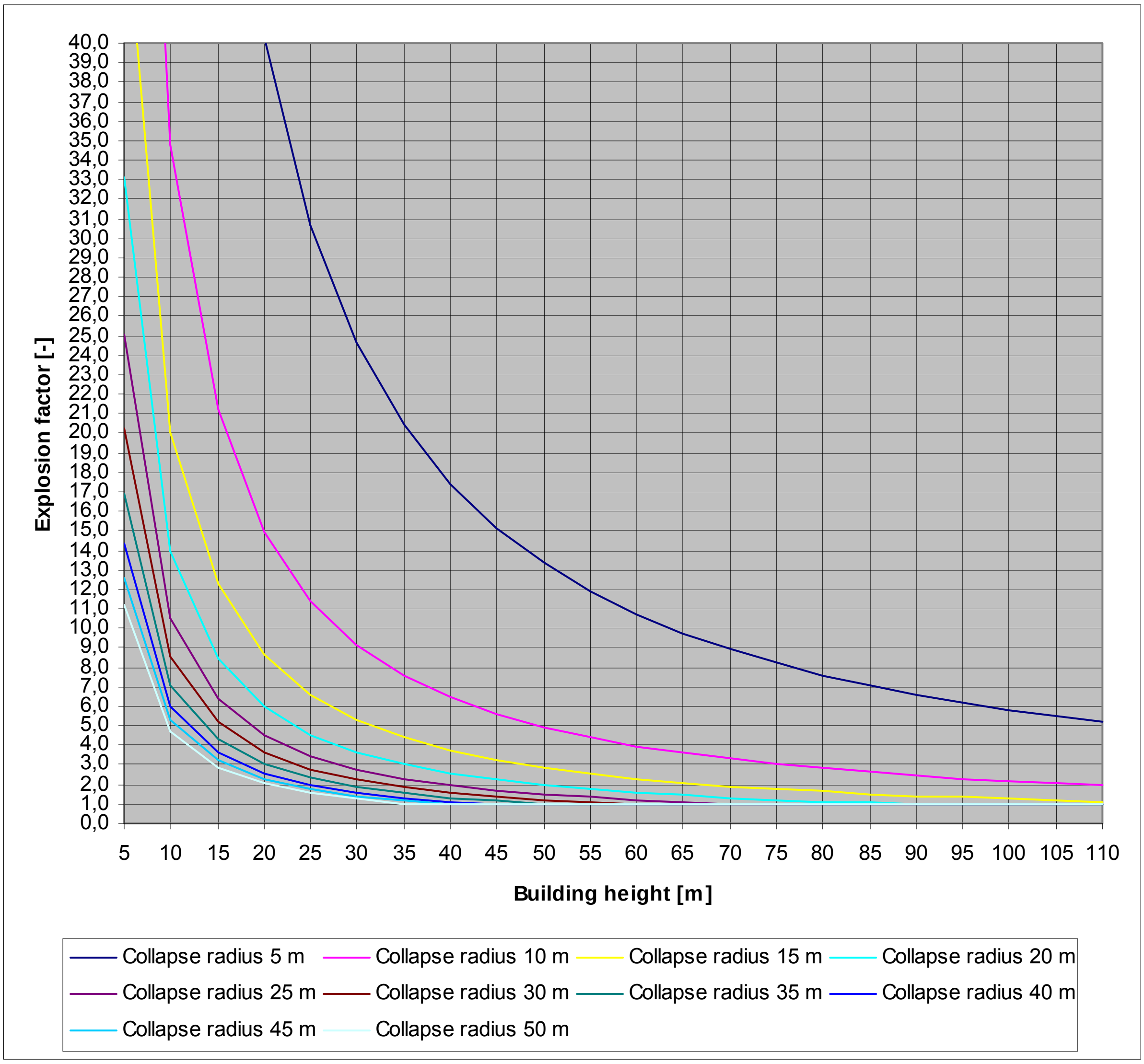


Figure 5-16: Graph explosion factor

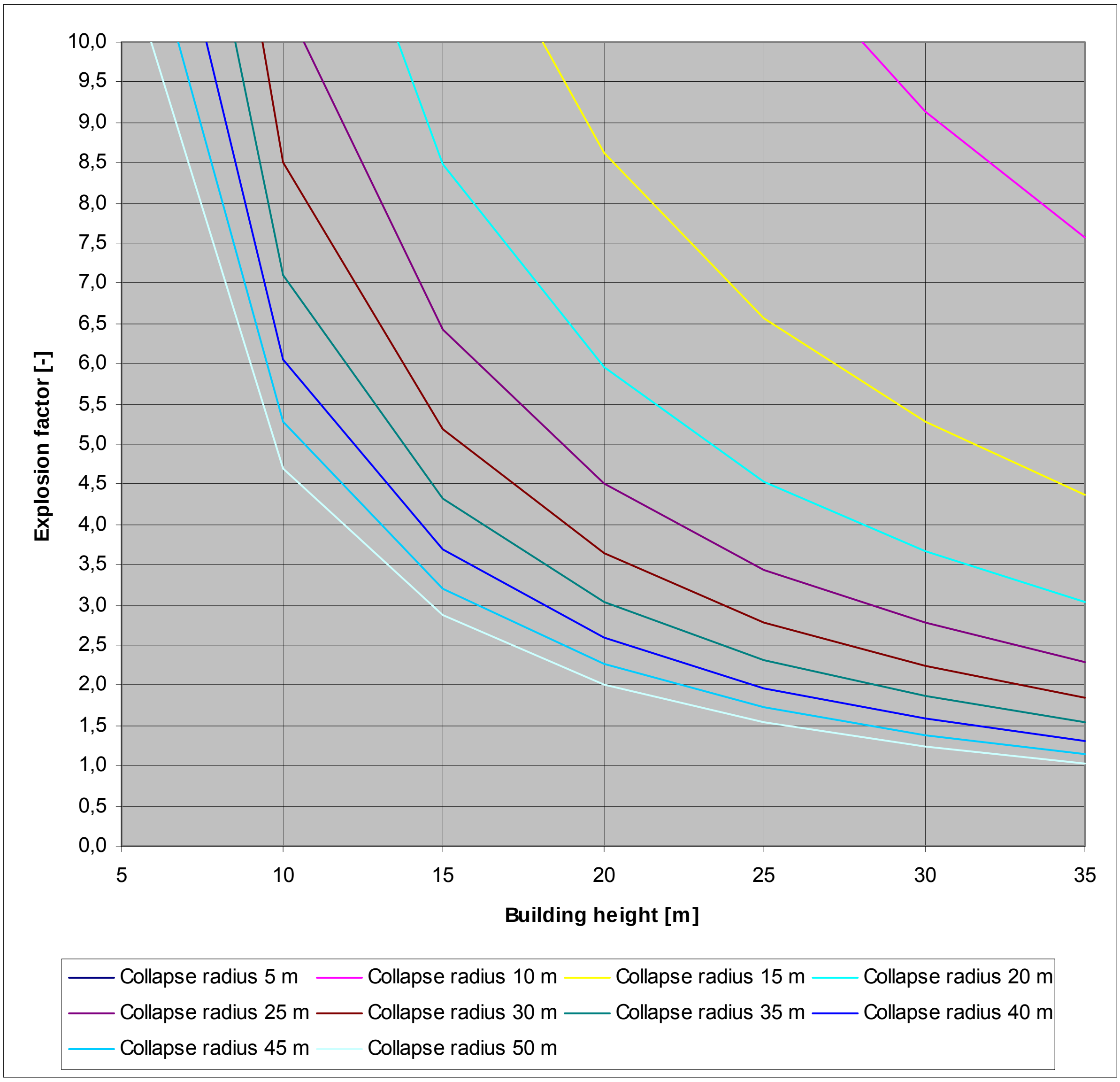


Figure 5-17: Detail graph explosion factor

This graph could be added to the code NEN 6702 to provide a tool for the blast resistant building design.

6 Conclusions and recommendations

6.1 Conclusions

1. The side on overpressure at a building 20 m from the road as a result of an explosion in three different scenarios was determined. These scenarios are in the tunnel 40 m down the road, in the tunnel mouth 20 m down the road and on the open road in front of the building. An explosion in the tunnel or in the tunnel mouth has a larger initial overpressure than an explosion on the open road, because the explosion takes place in a confined space. An explosion on the open road has a smaller distance from the building than an explosion in the tunnel or in the tunnel mouth. The hypothesis was defined that an explosion in the tunnel or in the tunnel mouth results in a larger side on overpressure at the building than the same explosion on the open road. Calculations of the side on overpressure has shown that the hypothesis is untrue. The largest blast load on a building and thus the governing load is from an explosion on the open road in front of the building and not from an explosion in a tunnel down the road. See table 3-3.

2. The most sensitive parameters of the blast load are the tank volume and the distance to the explosion. The most sensitive parameters of the mechanical properties of the building are the horizontal design load and the height of the building. A higher building results in a larger static strength and a smaller angular rotation, which reduces the damage. See tables 5-13 to 5-15.

3. Three methods were used to determine the damage as a result of a blast load. These methods are the chance of collapse, the equivalent static load and the required ductility. Sensitivity analyses of these methods that determine the damage to the global building and the structural elements have shown that the glass is the first to fail but the first structural elements to fail are the columns and walls perpendicular to the direction of the blast load. The distance from the explosion within which one type of element of a building (and thus the whole building) will fail is called the collapse radius.

	Collapse radius
Global building	58 m
Glass	363 m
Core	24 m
Floor	<1 m
Column	100 m
Perpendicular wall	83 m
Non perpendicular wall	9 m

Table 6-1: Collapse radii model building elements

4. The defined model building will collapse according to the three methods that determine the damage to the building as a result of a blast load. By changing the parameters of the elements of the model building the collapse radii of the elements can be reduced. Because the column and the perpendicular wall are the governing elements the parameters that should be changed are the column or wall width, the amount of reinforcement in the concrete and the concrete quality, decreasing the modulus of elasticity.

5. A building that will not collapse at a distance of only a few meters from the explosion is possible. In order to achieve this it is necessary that no walls perpendicular to the blast load, no separate columns and no beams are used. Furthermore, instead of using the wind load as the horizontal design load (in area

I according to NEN6702) a larger horizontal design load should be used, which is the wind load multiplied by an explosion factor which is introduced here. This explosion factor depends on the building height (just as the wind load) and the distance from the explosion (the collapse radius) which can be read from figure 5-17. The explosion factor is a tool for blast-resistant building design.

6.2 Recommendations for blast-resistant building design

1. This study gives a method to determine the response of a building to a blast load. The conclusions and recommendations are drawn up on basis of the sensitivity analyses of the different methods used to determine the response of the model building. For other buildings the dimensions and materials of the building and its elements will be different. When designing a building the response should be determined again.

2. A larger distance of the building from the explosion is the best way to reduce the load on the building, and thus enhancing the blast resistance. The distance from the road to the building should be maximized.

3. Increase the horizontal design load according to figure 5-15.

4. Use a round building geometry to reduce the reflection on the building.

5. Use a structure with a small angular rotation. The angular rotation is the radial frequency and is a measure of the rotation rate. A small angular rotation means a large natural period. High buildings have a larger natural period than low buildings. This means a high building is preferred over a low building. The angular rotation says something about how the building will react to the blast load. A low angular rotation results in a low equivalent static load and a low required ductility.

6. Because columns and beams have a low static strength in comparison with walls and floors do not use separate columns and beams, integrate them in floors and walls. Also don’t use shear walls perpendicular to the direction of the blast load. The direction of the blast load is the side of the building facing the road, that is where the explosion takes place. As the vertical load bearing and stabilizing structural elements only two type of elements should be used. These are walls in the direction of the blast load and cores. If columns are used they should not be placed in the façade of the building. The façade elements should transfer the blast load to the floors or non perpendicular walls and not to columns and beams. A column in the façade can have a 10 to 30 lower static strength than a column inside the building. Also if columns are used maximize the reinforcement and the depth in the direction of the blast load.

7. Use a lot of glass in the facade; it will reduce the reflection on the building. It will also enhance the danger for the people inside the building, but the flying around glass cannot be prevented. Windows of the model building will fail in a radius of 363 m. This distance mostly depends on the dimensions of the glass.

8. The building should be able to loose a large portion of the structure without total collapse. Make sure the building has a large redundancy. Use sacrificial elements and separate building parts. For the main load bearing structure it is important to always have a second load path to prevent progressive collapse. The failure of one column should not lead to the collapse of the whole building. A solution could be that buildings are designed to allow for the collapse of a complete façade or row of columns. If these fail the rest of building keeps standing. For this reason the

stabilizing elements or other main load bearing structural elements should not be in or near the façade of the building or on the side of road.

9. Increasing the dimensions of the core (maybe even a tube structure) does increase its blast resistance but because the core is not the governing element (see table 6-1) the building still can collapse.

6.3 Recommendations for future study

After the conclusions of this study there can be made recommendations for future studies following from this master thesis

1. Because the failure time of the tunnel structure is unknown it is not possible to predict the structural response of the tunnel to an explosion. Is the blast load time larger or shorter than the failure time? Possible solutions are hatches in the tunnel structure that can ventilate the overpressure. But the load time is probably shorter than the failure time which means the hatches will not work. The tunnel response, its failure time and possible structural solutions are recommended for further study.

2. Besides a positive phase duration a blast wave can also have a negative phase. The load of this negative phase on the building is negative. What is the effect on the buildings response of this negative phase duration of the explosion load?

3. Debris of cars and building parts or other elements destroyed in the explosion are projectiles against the building. What is the load of this debris on the building?

4. It was assumed in this study that the glass is brittle and will fail instantly and that in that case no reflection can develop and because of that the load on the building will be smaller. It should be studied if this assumption is correct. This effect of reduced overpressure on the building as a result of failure of the glass also applies for the other building elements. How does the failure time of an element effect the reduction of the reflected overpressure on the building and how can the failure time be determined?

5. The effect of shielding the building of by means of a barrier along the road should be studied. The barrier can be of concrete, steel or sand. A possibility of shielding is a lowered road. If the road is deepened and the buildings first floor starts 5 m above the road the maximum peak reflected overpressure and the total load on the building will be lower. Also the distribution of the load will be different.

6. Maybe it is possible that pressure waves occur in the columns, which result in a decrease of the load bearing capacity of the columns. This phenomenon is recommended for further study.

7. The largest horizontal wind design load is on the top of the building and the smallest wind load is on the foot of the building. In the method to determine the static strength of the building and its elements this non uniform load distribution is taken into account. The overpressure as a result of the blast has a reverse distribution; the reflected peak overpressure at the foot of the building is larger than the pressure at the top (see figure 5-12). For the damage to the elements of the model building this reverse distribution of the blast load is taken into account. Paragraph 5.4.4. showed that a considerable reduction of the equivalent static load can be obtained. The equivalent static load on a column in the model building of 20 m wide and 35 m high is 494 kPa at the foot of the building perpendicular to an explosion 20 from the building and 186 kPa at the upper corner of the building.

But for the damage to the global building this non-uniform blast load is not taken into account. For the global building a uniform equal distributed load on the building façade is assumed of the maximum reflected overpressure at the foot of the building. This is a very conservative approach. Furthermore it is unknown how the building will react to the non-uniform load. Further study is recommended.

8. A higher building increases the design load and decreases the angular rotation. But several high buildings can form such a dense building area that the reflections increase and a sort of tunnel effect occurs. Further study is recommended.

9. This study only focused on the prevention of collapse of the building. In the used methods it was assumed that if one element fails all elements of the same type will fail, because they all have the same dimensions, which results in collapse of the whole building. Or in other words: if one wall fails the wall next to it will also fail because it has the same dimensions and approximately the same load. It is not necessarily true that if one element fails all other elements will also fail. It is possible that only a part of the building collapses. It is unknown how many elements initiate progressive collapse or even total collapse. The progressive collapse of each building is unique and therefore hard to predict. It is unknown how the blast wave travels true the building structure. The relation between the amount of lost elements and the effect on the building structure integrity should be studied.

10. Are the methods used to determine the explosion load, the mechanical properties and the damage to the building correct? How reliable are the outcomes? Most methods are analytic and some are empirical. It is recommend to perform a validation of the used methods. It is suggested that a FEM analysis is performed to validate the used methods that describe the mechanical properties of the building.

11. In the paragraph where the difference of the response of a round building and a rectangular building is discussed no attention is paid to the height of the building. The reflection coefficient and distance also change with the height of the building and therefore attribute to the load on the building. Further study is recommended.

12. The method and solutions in this study are based on a concrete building. The response of a steel or wooden building also should be studied.

13. It was assumed in this study that the collapse of the main load bearing structure would result in the most injury to the people. But maybe it is true that the flying around glass is more harmful and results in more injury.

14. As stated a blast-resistant building can be designed at only a few meters from the explosion. This study focused only on the structural response of the building which resulted in this conclusion. It is expected that at these extreme close ranges other effects will result in large damage to the building. Further study is recommended.

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