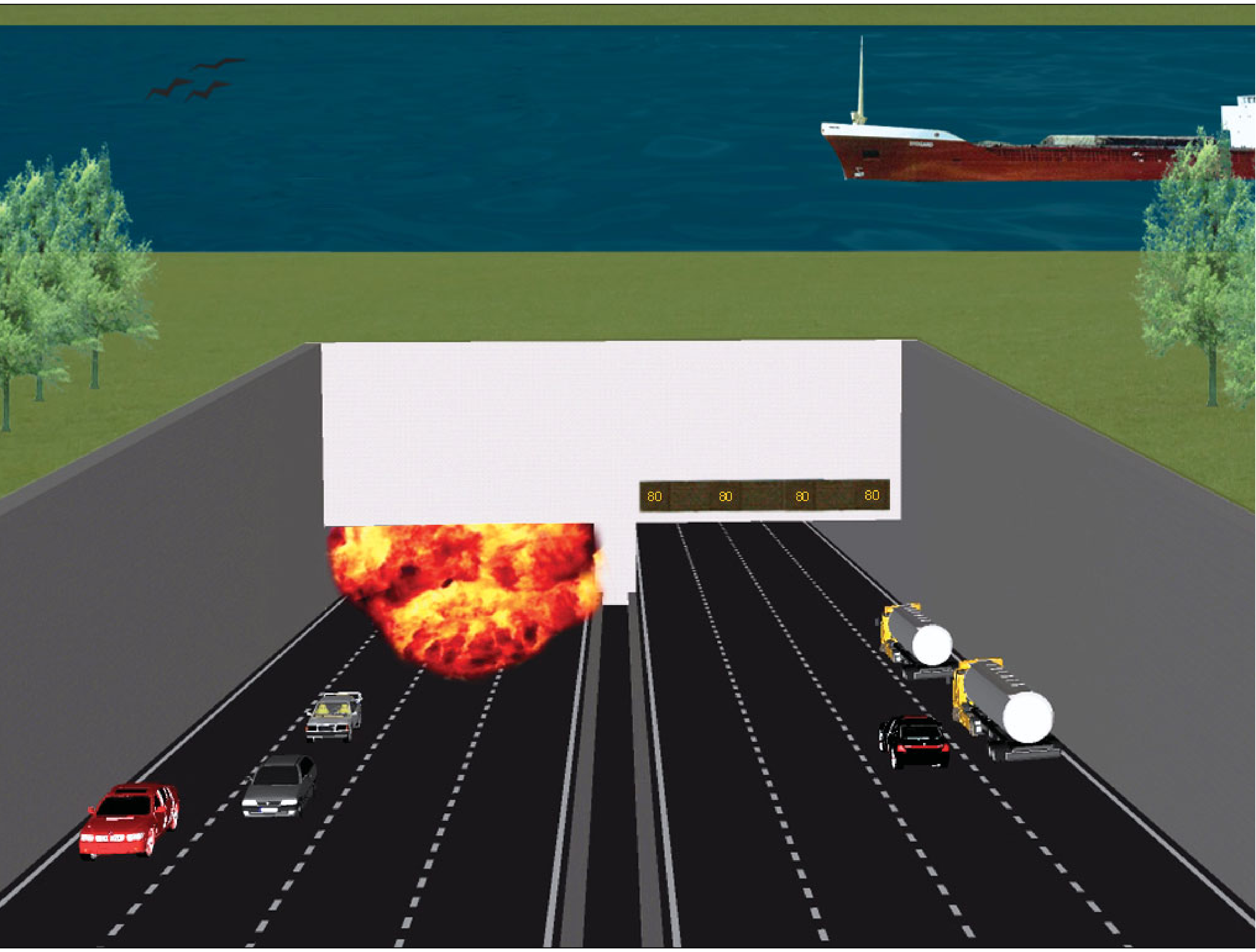
Explosion loads in immersed tunnels

February 2009

D.J. de Jong



Delft University of Technology

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Preface

This thesis is the result of research performed in the final stage of the Master curriculum of the master variant Hydraulic Structures, faculty of Civil Engineering and Geosciences at Delft University of Technology. The research is carried out in cooperation with BAM Infraconsult bv, Gouda, The Netherlands. This document provides relevant information and conclusions drawn from the performed investigation. I would like to extend a word of thanks to the thesis committee for their supervision and support.

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Abstract

Certain goods that are transported via roads have properties that may cause severe effects in case of an accident. The most striking events are the occurrence of fire and explosions. Since tunnels are isolated environments, the risks inside are considerably higher than for the open field. If an explosion takes place inside a tunnel, many casualties and severe damage are to be expected. Although the probability for such an event is rather small, restrictions concerning transport of goods that are potentially dangerous in this respect apply. Because of these restrictions, alternative routes have to be used, often resulting in long detours sometimes crossing built-up areas, which is undesirable. Therefore, there are situations in which it is desired to ease the restrictions for transport of dangerous goods trough tunnels.

Recently, for an immersed tunnel project in Antwerp, the Oosterweel tunnel, a requirement concerning the explosion resistance was stated. Implicitly this requirement means that in the event of an explosion casualties are accepted, while the structure should not collapse and repair is possible, in order to prevent great economical loss. The requirement stated exists of a static pressure of 500 kPa and a suction of 300 kPa that have to be accounted for in the structural design. The nature of these requirements is not clear however.

During the tender phase of the Oosterweel tunnel, BAM Infraconsult concluded that this requirement has large influence on the required thicknesses for the elements of the immersed tunnel. During transport, these concrete elements should float, while in the final situation sufficient stability in order to prevent uplift has to be provided. These are contrary demands that result in a very delicate balance, which is characteristic for an immersed tunnel. The requirement for the explosion load interferes with this balance and makes it difficult to comply with all demands efficiently if traditional design methods are applied.

This research is at first initiated in order to evaluate the stated statically defined requirement concerning explosion loads for the Oosterweel tunnel. Secondly, possibilities for adaptations in the design in order to provide the required capacity in a efficient way are explored. From literature study, it became clear that the relevant design codes provide barely any information concerning explosion loads. There is however research available that can be used. From the results of this research and in consultation with experts, it is concluded that the representative explosion load is due to a LPG BLEVE. The order of magnitude that has to be taken into account is obtained from recent research into this topic by TNO.

In order to investigate the structural response of an immersed tunnel, an analytical model is developed, whereby the cross-section is divided in several elements that are schematized as single degree of freedom mass spring systems that are exposed to a dynamic load. The results that are obtained with this model are comparable to the findings of TNO using a more complex numerical model. Therefore, the developed analytical model is a suitable screening tool, although there are a few limitations and simplifications. Subsequently, the dynamic module of the finite element code Plaxis is used to investigate the response of the structure to a dynamic load. This more advanced model enables to take the effect of soil and water into account.

From calculations with the described models, it is concluded that the statically defined requirement stated for the Oosterweel tunnel does not comply at all with the representative BLEVE load according to TNO. Although the requirement for the Oosterweel tunnel has already a large influence on the design of the elements it is still of much too small magnitude.

Subsequently, possibilities to adapt the design of the tunnel in order to provide sufficient capacity to withstand the representative explosion load according to TNO are investigated. Several solutions, among which reduction of the load, dissipation of energy and application of alternative materials are judged qualitatively. Two solutions are selected for further research.

At first, a sandwich structure is investigated, since it was considered a potentially beneficial solution. Apart from that, the principle is interesting from a scientific point of view. Exploring calculations regarding the resistance against explosion loads are performed. Besides this, the stability during transport as well as in the final situation is considered. It is concluded that it is technically possible, though not easy to realize this solution. Besides this, the solution appeared not as effective as expected regarding the resistance against explosion loads.

Secondly, the application of separate tubes for the transport of dangerous goods is evaluated. It can be concluded that this alternative provides a suitable solution that is technically possible and realistic. In case this solution is applied, the risk involved with transport of dangerous goods passing a tunnel will be reduced to a large extent. The vehicles transporting the dangerous goods are separated from the regular traffic. Therefore, the probability of an accident decreases and the consequences in case of explosion will be less severe. The adaptations will result in significantly increased costs compared to a regular tunnel, though there are possibilities with respect to traffic management and the use of the special tubes that make this alternative more attractive in an economical sense. In case an explosion resistant tunnel is desired, it is recommended to consider this solution.

Table of contents

1 Introduction to the research

This chapter provides a description of the problem, as well as a problem statement and objective for the research. Apart from that, the structure of the report will be explained.

1.1 *Problem description*

Some goods or materials that are of major importance for a developed area have properties that may cause undesired effects in case of an accident. Transport of these dangerous goods is necessary for the economical activities however. In case of an accident, the presence of dangerous goods may cause severe fire or even explosions, resulting in damage and possible casualties. For tunnels, which are relatively isolated environments, the consequences of an accident with dangerous goods are more striking than in the open field. Therefore, restrictions apply concerning the amount and nature of goods that are allowed to be transported trough tunnels. Some goods are prohibited in certain tunnels, leading to relatively long alternative transportation routes, sometimes passing the built-up areas, which is undesired. Limitation of the risk involved with transporting dangerous goods through tunnels can be achieved by taking measures to reduce the probability of an accident. Furthermore, effect limiting measures can be taken. In case of a normative explosion, it is assumed that none of the people that are inside a tunnel at that moment will survive. Therefore, these measures will only concern the limitation of economical loss. For intensively used tunnels, with a relatively large transport of dangerous goods, this might be desirable. In that case the tunnel should not collapse and it should be achievable to repair the structure after an explosion. Due to the extreme loads that occur in the event of an explosion, the structure should be of large dimensions and significantly increased amounts of materials are required. Especially for immersed tunnels, a delicate balance between the deadweight and volume of the elements exists. Proving sufficient capacity to withstand explosion loads leads to several problems with respect to design and execution. Since an immersed tunnel is often an attractive alternative for crossing a waterway, a solution has to be sought for. BAM Infraconsult was confronted with an extraordinary requirement concerning explosion loads in the tender phase of a recent project. The traditional design method results in large dimensions and the requirements for the transport phase could be barely met. In order to provide more insight into the effect of explosions in tunnels and explore solutions, this research is initiated.

1.2 *Problem statement*

If it is desired to design an immersed tunnel that has sufficient capacity to withstand explosion loads, the structure should be of excessive dimensions, if it is even possible, in case traditional methods are applied.

1.3 *Objective*

Investigation of the representative loads and structural response to explosion loads of an immersed tunnel. Apart from that, exploring the application of structural measures in order to withstand explosion loads in immersed tunnels effectively, whereby both the technical and executive aspects will be considered.

1.4 *Specific goals*

For BAM Infraconsult, the following aspects are of interest.

- Evaluation of the extraordinary requirement concerning explosion loads as stated in a recent project, the Oosterweel tunnel.

- Investigation of alternatives and practical solutions for an explosion resistant immersed tunnel.

- Consideration of the design aspects as well as the consequences for the execution phase.

1.5 *Starting points*

The research is framed in order to focus on the most relevant aspects. A number of starting points is stated in collaboration with BAM Infraconsult and are listed below.

- The research will focus on immersed tunnels. In order to justify the relevance and importance of this concept, in chapter 2 the major alternatives that are available for the crossing of a waterway will be discussed.
- The emphasis of the research will be on explosion loads. An explosion is the most striking event that may threat the structural integrity of an immersed tunnel. Since these loads in many cases are strongly connected to the occurrence and development of fire, it is inevitable to consider this phenomenon as well.
- It is decided to consider immersed tunnels for road traffic, since these are more common than rail tunnels. Furthermore, the majority of dangerous goods are transported via roads. In general the results from the research will be applicable for rail tunnels as well.
- Since reinforced concrete elements are commonly applied for immersed tunnels in the Netherlands, it is decided to emphasize on this variant.
- BAM Infraconsult stated that it is desirable to make as few adaptations to the principles and methods for immersed tunnels as possible, since a lot of experience is gathered and the processes are standardized to a great extent.
- The purpose of this research is not to produce complete designs. It should be considered as an exploring feasibility study, whereby the emphasis will be on the structural aspects.
- Innovation is an important motive for this research. Therefore, solutions which may be less attractive in an economical sense may be investigated anyway.

1.6 *Structure of the thesis*

The contents of the next chapters are briefly explained in the following.

In chapter 2, the framework and relevant background information to this research is presented. The contents are established from literature study and in consultation with BAM Infraconsult.

Chapter 3 provides an overview concerning the regulations that apply for structural safety of a tunnel. This chapter is the result of literature study and the interpretation of research that is relevant in this respect.

In Chapter 4 the structural response of an immersed tunnel to an explosion load is investigated. This is done by means of analytically dynamic calculations, whereby the elements of the cross-section are schematized as mass spring systems. Furthermore dynamic calculations with the software package Plaxis are performed. Subsequently the requirement stated for the Oosterweel tunnel is reviewed. This requirement was the reason for the initiation of this research.

Chapter 5 provides an introduction to the investigation of possibilities for designing an explosion resistant immersed tunnel. A representative cross-section and starting point are determined. Furthermore the effect of the requirement stated for the Oosterweel tunnel to the design of a tunnel is investigated.

In chapter 6 an overview of ideas and possible solutions concerning the design of an explosion resistant immersed tunnel is presented. A few alternatives are selected for further investigation.

Chapter 7 deals with two alternatives that are considered to be of interest for further research. The effectiveness and structural feasibility of a sandwich structure as well as the application of special tubes is investigated.

In chapter 8 the most important conclusions and recommendations resulting from this research are presented.

2 Problem Analysis

This chapter provides relevant background information for the research. The framework of the research and starting points originate from the considered aspects. To start with, the alternatives for the crossing of a waterway will be discussed. Subsequently, attention will be paid to transport of dangerous goods. Furthermore, background information concerning the Oosterweel tunnel will be provided, since the requirements concerning explosions for this project triggered the research.

2.1 *Crossing of waterways*

As a result of the fast economic development all over the world, there is an increasing need for infrastructure in order to improve the connectivity. This is especially the case for delta areas like the Netherlands, where industries and ports expand rapidly. Crossing a waterway requires special attention, since many aspects are involved. There are several alternatives to cross a waterway. A number of possibilities will be discussed briefly.

2.1.1 Flexible and permanent connections

In case of low intensity of the traffic, a ferry connection may be suitable. The advantage of this solution is the flexibility to adjust the capacity in order to comply with the changing demand for transport. The capacity is however limited and if higher traffic intensities are expected, which is very likely in highly developed areas, it is not a very suitable option. A permanent connection provides a better solution. Two main alternatives are a bridge or a tunnel.

2.1.2 Bridge

A bridge provides a good solution for road or rail traffic. Though, if a navigation route is crossed, this alternative causes a limitation for the height of the ships. Taking the lifetime of the structure into account, in combination with the increasing size of vessels during the last decades, this is an important disadvantage.
In case a moveable bridge is chosen, there is no limitation for the height of ships that can pass. There will be delays for both the road traffic and ships however, since these flows can not proceed simultaneously. For a connection that will be used intensively, this is unacceptable.

2.1.3 Cut and cover tunnel

An in situ tunnel is constructed in two stages. During the first stage, approximately half the width of the waterway is closed by means of a cofferdam. This dam is commonly edged by means of sheet piles. Usually an underwater concrete floor is applied. Inside this pit, the first part of the tunnel is constructed. If the first part is completed, sheet piles are removed and from the other bank the same procedure is starts.
Crossing a waterway by means of an in situ tunnel is very uncommon in the Netherlands. The main reason for this is the severe impact on the landscape during construction, as well as the considerable hindrance for navigation and the obstruction for the discharge of rivers. Due to these disadvantages, this alternative is no further considered within the framework of this report.

2.1.4 Bored tunnel

It is also possible to create a tunnel in situ, by means of a tunnel boring machine (TBM). Initially this method was only applied in hard soils like rock. Recently, a lot of experience is gathered with boring tunnels in soft soil and tunnel boring machines are very sophisticated nowadays. In figure 2-1 a principle scheme for a TBM is depicted.

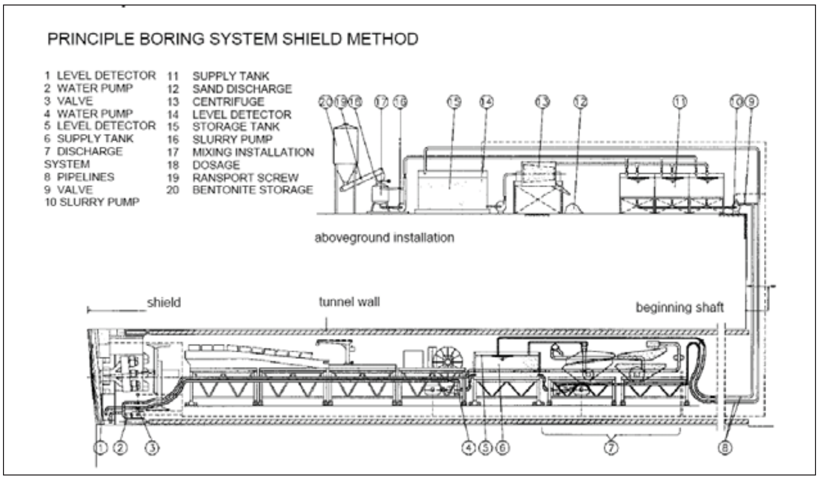


Figure 2-1 Principle scheme for a TBM [22]

Excavation is done by means of a rotating cutting wheel. It is important to create a stable excavation face. There are several methods available to achieve stability. For the subsoil conditions in the Netherlands, the face can be supported by means of air pressure, bentonite pressure or ground pressure.

The construction method leads to a circular cross-section. The lining is composed of several prefabricated concrete elements, which are positioned and placed by the TBM. The TBM is equipped with jacks that push against the already constructed rings in order the move forward. A typical cross-section of a bored tunnel is depicted in figure 2-2.

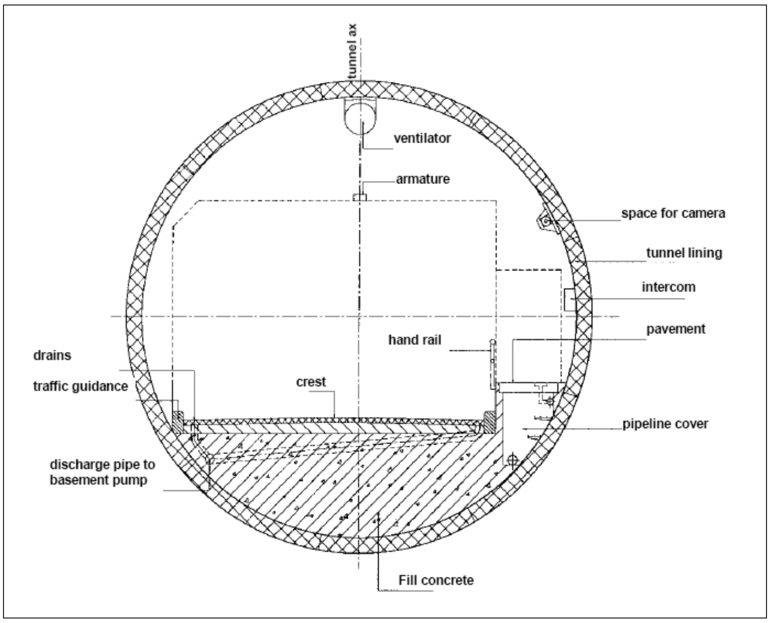


Figure 2-2 Typical cross-section of a bored tunnel [22]

The boring process starts in a departure shaft where a support frame is present, to resist the jack forces of the moving TBM. A few blind rings are constructed and the TBM starts drilling through a low strength concrete sealing block. If this block is passed, the process continues as described before. The arrival shaft is also sealed with a low strength concrete block, in order to provide stability.

2.1.5 Immersed tunnel

One method to construct a tunnel is based on the prefabrication of several elements, that are transported floating to the desired location and will be immersed there. The cross-section of an immersed tunnel can have various shapes and the elements can be made of steel or reinforced concrete. In the Netherlands usually reinforced concrete is applied, a typical cross-section of an immersed tunnel is depicted in figure 2-3.

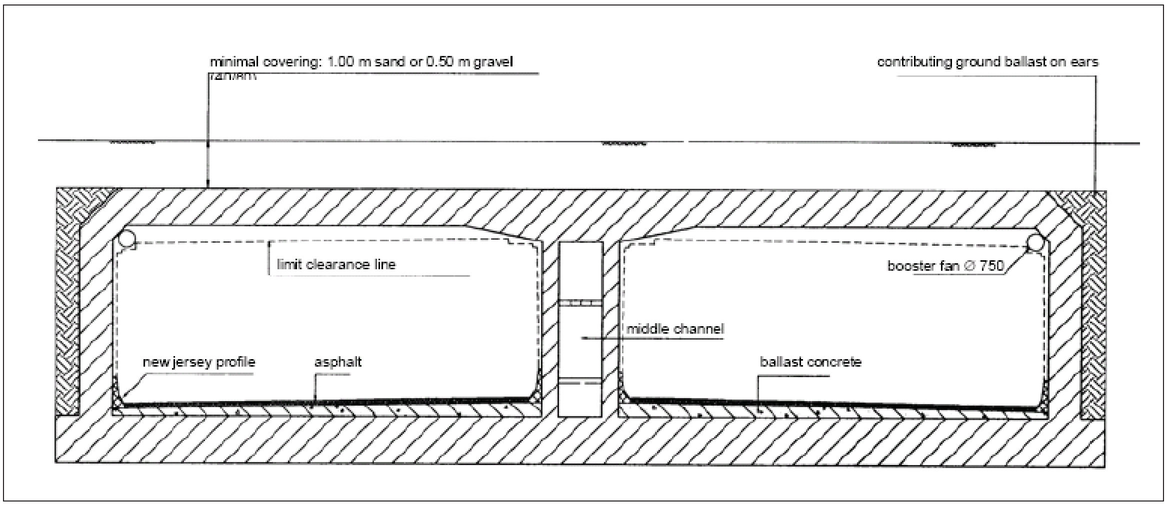


Figure 2-3 Typical cross-section of an immersed tunnel [22]

Prefabrication of the elements can be done in a construction dock for instance. Depending on the local circumstances existing docks or new ones in the vicinity of the project site were used for recent projects in the Netherlands. The elements will be immersed and connected to each other in a dredged trench and covered with a relatively thin layer of soil. At the banks of the waterway, abutments are made which form the transition between the road and the immersed tunnel. These abutments provide a stable base that supports the tunnel elements. There are several methods for the construction of immersed tunnels and a lot of experience is gathered in The Netherlands during the last decades. Characteristic for immersed tunnels is the strong interaction between the design and the construction method as schematized in figure 2-4. The design is influenced by boundary conditions and functional requirements concerning the final situation. The tunnel should be stable on the bottom, for instance which implies that sufficient deadweight is required to prevent uplift. During transport, the elements should be floating, however. These requirements are contrary, but with the application of ballast concrete, a suitable solution can be found usually. The range in which the dead weight can be varied is very narrow although.

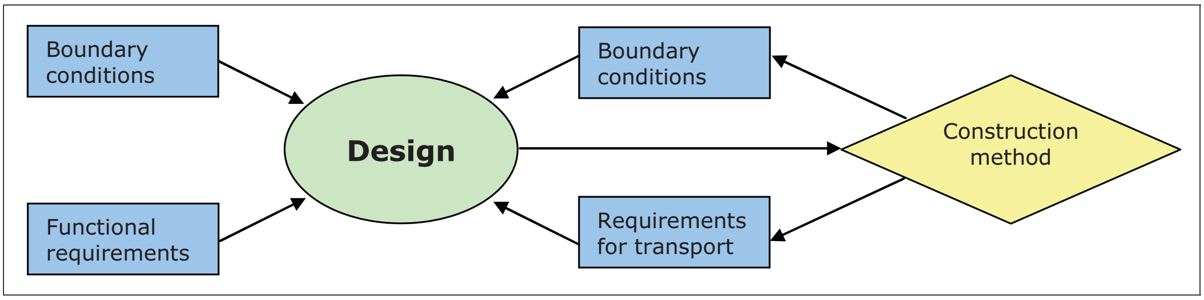


Figure 2-4 Interaction between the design and construction method

2.1.6 Comparison of the alternatives

Many waterways in The Netherlands are intensively used for navigation. The size of vessels increase rapidly these days and considerable navigation heights are required. In order to facilitate sufficient free space, bridges have to be situated further beyond the banks. If a waterway is crossed by means of a tunnel, traffic and navigation are completely separated. This leads to an optimal situation for both flows. A bored tunnel should be covered with a layer of soil with a thickness in the order of the diameter of the tube, in order to prevent uplift. A deeper tunnel results in a greater length, due to restrictions for slopes and sight lines. For an immersed tunnel a cover in the order of 0.5 – 1 m is sufficient to provide stability.

Generally it can be stated that an immersed tunnel requires a relatively small length compared to a bridge and a bored tunnel, this is depicted in figure 2-5.

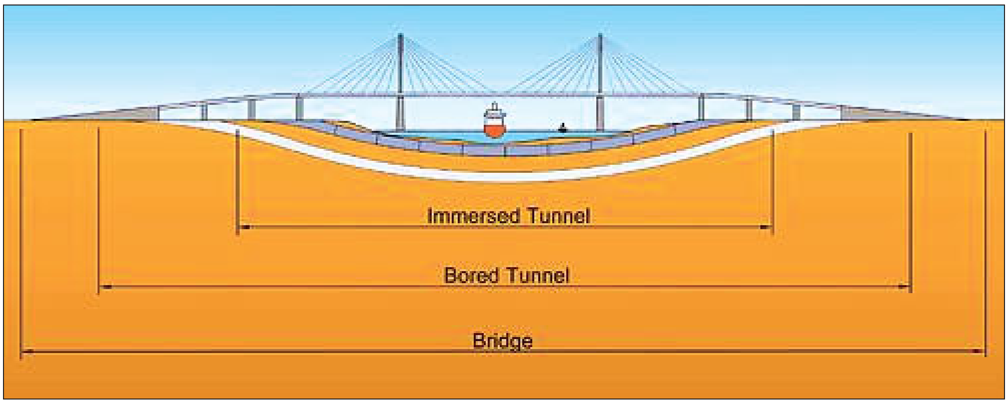


Figure 2-5 Comparison of the required length for different crossings [25]

A Bridge should be of relatively large dimensions in case an intensively used navigation route is crossed. Bored tunnels become more and more attractive due to improvement of the technique and gathered experience. In general it can be stated however that for the Dutch circumstances an immersed tunnel often leads to the most economical solution. Furthermore, a lot of experience is gathered with this construction method during the last decades. Therefore, the processes are optimized and risks are well known. From the foregoing it is clear that immersed tunnels are of major importance for the Dutch infrastructure. Therefore, it is justified to focus on this type of crossing within this research.

2.2 *Transport of dangerous goods*

Transport of materials and products is of major importance for highly developed areas. Especially in the Netherlands, the transit of goods is an importantly economic activity due to the presence of one of the largest ports in the world. Some of the goods that have to be transported can be dangerous for humans, animals and the environment however. Therefore, strict regulations apply concerning the handling and transport of these goods. The following paragraphs provide detailed information related to the situation in The Netherlands.

2.2.1 Classification of dangerous goods

The regulations concerning transport of dangerous goods in Europe are stated in the Accord européen relatif au transport international des marchandises Dangereuses par Route (ADR). According to this document, the following dangerous goods can be distinguished [21].

Class 1: Explosive objects or materials
Depending on the goods, there can be danger of massive explosions, fragments, fire and high pressures. This category includes for example ammunition, civil explosives and fire works.

Class 2: Gasses
This class includes gasses that are compressed, liquefied and liquefied under pressure. Depending on the particular gas, there may be danger of fire, asphyxiation, intoxication, oxidation or corrosion. Examples of gasses that belong to this class are LPG, oxygen and chloride.

Class 3: Flammable liquids
This class includes flammable liquids with a boiling point up till 35 °C that are also toxic. Furthermore, flammable substances with a boiling point higher than 35 °C and an ignition temperature lower than 23 °C. The third type concerns liquids with a boiling point higher than 35 °C and an ignition temperature between 23 °C and 60 °C. Examples are ink, paint and gasoline.

Class 4.1: Flammable solids
This class includes products with a melting temperature that is higher than 20 °C. Besides this, explosive materials that are modified in such a way that these are no longer explosive are covered in this class, as well as self-decomposing materials. Examples of products are matches, sulfur and metal powders.

Class 4.2: Materials that are sensitive for self-ignition
This class concerns materials, including mixtures that will start burning within 5 minutes after exposure to air. Furthermore, materials that are sensitive for self-heating belong to this class. Examples are phosphor, cotton disposal from garages and silt from sewers.

Class 4.3: Materials that develop flammable gasses if exposed to water
This class concerns materials that develop flammable and sometimes explosive gasses, because of a chemical reaction with water. Aluminum and magnesium powder as well as calcium carbide are examples of material that are covered within this class.

Class 5.1: Oxidizing materials
Oxidizing materials might be not flammable at all, but these materials can stimulate the development of a fire. Water peroxide and chili saltpeter are examples of this class.

Class 6.1: Toxic materials
This class covers materials that cause injuries or even death after inhalation, swallowing or skin contact. Good examples are chloroform, cadmium and pesticides.

Class 6.2: Materials that cause infections
These are materials that contain micro organisms, including bacteria, parasites and fungus that cause diseases to humans and or animals. Disposal from hospitals is an important example of this class.

Class 7: Radioactive materials
This class concerns goods that are dangerous for radioactive radiation and contamination. Waste material from a nuclear power plant and uranium belong to this class for example.

Class 8: Corrosive materials
These materials affect the upper skin. Furthermore, other materials in the vicinity may suffer from corrosion, leading to dangerous situations. Sulphuric acid, battery acid and hydrochloric acid are examples of this class.

Class 9: Remaining dangerous goods
This class concerns materials and goods that are dangerous in a way that is not covered within the foregoing classes. Examples are asbestos and airbags modules.

2.2.2 Regulations for tunnels in the Netherlands

Approximately 10 % of the entire amount of transported goods in the Netherlands can be classified as dangerous. Transport of dangerous goods takes place by road to a large extent, as can be seen from figure 2-6. In the period 1990-2002 there were hardly any changes in this distribution and therefore it is considered to be still representative [2].

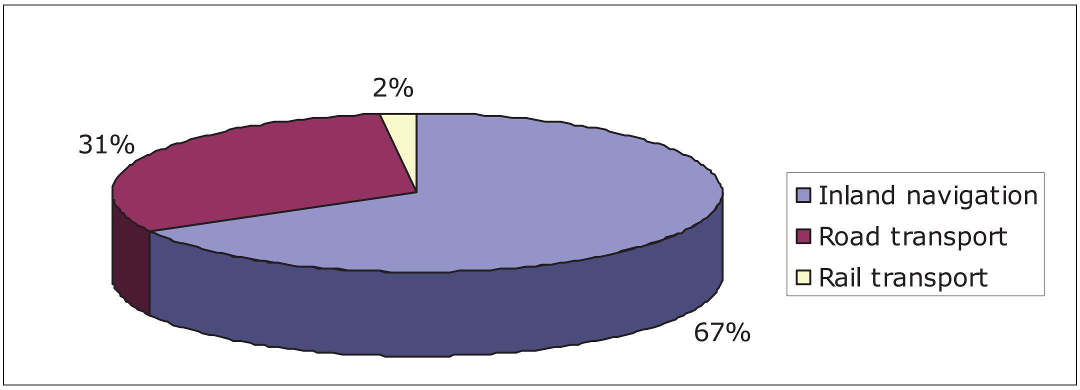


Figure 2-6 Transport of dangerous goods in The Netherlands

Especially for tunnels, the effects of dangerous goods in case of an accident may be catastrophic. Since a tunnel is an isolated environment, the consequences of fire, the release of gasses, explosions etc. can be severe, resulting in injuries or even human loss. Besides this, the structural integrity may be endangered in case of an accident, resulting in even more casualties and economical loss. Therefore, very strict regulations concerning the transport through tunnels apply.

2.2.3 Current regulation

In the Netherlands, tunnels are classified at this moment, three categories are distinguished which are listed below. For each category some examples are given.

Category 0: No restrictions compared to the road

- Schipholtunnel

Category I : Explosive goods are prohibited

- Beneluxtunnel
- Coentunnel
- Drechttunnel
- Noordtunnel
- Vlaketunnel
- Wijkertunnel
- Zeeburgertunnel
- Westerscheldetunnel
- Caland tunnel (Thomassentunnel)
- Sijtwendetunnel

Category II: Explosive goods are prohibited as well as flammable goods

- Botlektunnel
- Heinenoordtunnel
- IJtunnel
- Kilttunnel
- Maastunnel
- Piet Heintunnel
- Velsertunnel

For every new tunnel, a decision concerning the category is made, based on a risk analysis. In such an analysis, the safety for the tunnel itself is considered, as well as the safety of the surroundings. Probabilities for accidents and the consequences, like economical loss and possible casualties are quantified. Furthermore, protective measures in order to limit the consequences or the probability of an accident can be applied, although this will result in higher costs. The final decision will be based on the most economical solution including all mentioned aspects for the particular situation. In other words, the purpose is to determine the optimal, acceptable degree of safety.

For each category, there are detailed requirements concerning the loading, packing, handling and transport of the dangerous goods that are allowed. Furthermore, requirements concerning the vehicles and the safety equipment apply. The crew of a vehicle that transports dangerous goods should be qualified to do so and comply with a number of safety rules. These regulations are meant to limit the probability of an accident. For relatively small amounts of dangerous goods, exceptions may apply.

2.2.4 Regulation in the near future

Since 1 July 2007, there is a new classification for tunnels according to the ADR. In December 2009 this will apply in all countries where the ADR is used. Since a lot of border crossing transport takes place in Europe, it is desirable to use one system for classification.

All tunnels will be categorized in five different classes, which are defined below.

Category A: No restrictions
Category B: Restrictions for goods that may cause a massive explosion
Category C: Restrictions for goods that may cause a massive explosion or may release toxic gasses
Category D: Restrictions for goods that may cause a massive explosion, toxic gasses or fire
Category E: Restrictions for all dangerous goods, with only a few exceptions

For an overview of the goods that are restricted for each particular category, reference is made to [21], for clarity. Many exceptions apply within the classification as given in paragraph 2.2.1. and there is an extensive system to categorize all goods. Distinction is made between packed goods and materials that are transported by means of a tank.

2.2.5 Consequences of the regulation

The fact that dangerous goods are not allowed to be transported through tunnels results in several negative effects in a practical and economical sense. Since tunnels are usually part of the main road network, prohibition of dangerous goods will lead to detours and increased transport time. According to the Dutch traffic law, a driver of a vehicle loaded with dangerous goods should avoid built-up areas. Therefore, it is even more difficult to find a suitable route. If it is however impossible or not reasonable to avoid the built-up areas an exception can requested for to the authorities. It can be concluded that the strict regulations result in long transport times and result in inefficient transport of the particular goods. From this point of view, it would be desirable to have no limitations for the transport of goods via tunnels at all.

2.3 *Undesired events*

For a tunnel, there are numerous undesired events, both of minor and major importance. This chapter gives an overview of the most striking undesired events, since these are of interest for the design of the tunnel and policy concerning dangerous goods.

2.3.1 Fire

Fire can be generally described as a combustion that can propagate unhindered and cause damage and or danger. A fire can be started on purpose, this called fire-raising. Often it is however the result of an accident. Three factors are of major importance for the start and development of a fire. These factors are the presence of flammable material, oxygen and a temperature that is high enough for ignition. Besides this, the mixture rates and presence of a catalyst are also of importance.

If a fire occurs inside a tunnel, several phenomena threat the safety of people inside. The increase of temperature may cause burns. Furthermore, smoke will develop, which can lead to asphyxiation and limited visibility. These aspects are extreme within the isolated environment of a tunnel and escaping the fire will be complicated. Because of the high temperatures that may develop, also the structural safety of the tunnel may be endangered. The concrete may burst and sputter. Apart from that, the strength of the reinforcement steel starts to decrease at a certain temperature. In case of dangerous goods, a fire can also lead to an explosion, as indicated in paragraph 2.3.2. The probability of occurrence and the involved consequences of fire in a tunnel is of such order of magnitude, that usually a number of preventive and effect limiting measures are taken.

2.3.2 Explosions

An explosion can be described as a sudden increase of the volume of a medium, resulting in the release of energy, leading to temperature rise and the release of gasses. An explosion causes shockwaves, and the effects to the surroundings can be severe. Explosions can have several causes, both natural and artificial. An example of a natural cause is volcanic activity, while artificial created explosions for instance occur as a result of the use of weaponry.

2.3.2.1 Gas explosion

For the transport of dangerous goods through tunnels, the occurrence of a gas cloud explosion should be considered. In case of an accident, vehicles may be damaged leading to leakage of the material that is transported. If a pool with a dangerous liquid develops, this may lead to vaporizing, resulting to a gas cloud. If the gas is flammable, and the mix ratio with oxygen is between certain boundaries, ignition will result in an explosion. In case the liquid is also flammable, furthermore, intense fire is to be expected. It is also possible that gas is continuously released from the container as a result of an accident. In case of instantaneously ignition, a torch like fire may occur. In case a cloud can develop before ignition however, depending on the speed of the ignition and the propagation velocity of the flames an explosion may occur.

Deflagration
If ignition of a gas cloud occurs, the flame front initially develops spherically. From the moment the cross-section is completely filled, the front propagates towards the tunnel entrances, initially with a speed of only a few meters per second. The wave font is however the transition between the combusted and non combusted media. Since there is a large difference in temperature between these media, considerable expansion occurs, resulting in a current that carries along the flame front. Depending on the direction of the pressure gradient, instability of the flame front will occur, leading to an increased flame surface. As a result, the combustion speed increases and therefore the propagation speed of the flame front. Further acceleration takes place since velocity gradients develop as a result of geometry of the tunnel tube and the presence of vehicles. The increasing velocity also results in more intense turbulence and the flame front transforms into a mixture of combusted and non combusted media. If this process continues a deflagration may under certain circumstances turn to a detonation.

Detonation
As a result of the very turbulent mixture of hot reaction products and cold non combusted gas, locally extinguishing of the flame front may occur. A very reactive mixture consisting of combusted and non combusted media will develop and a sudden temperature rise may lead to a sub explosion that generates a blast wave that reinforces the flame propagation to a great extent. For practical applications, TNO defined a gas detonation in [11] as a gas explosion whereby the flame front is driven by a compression wave. This shockwave compresses the flammable medium far beyond its self ignition temperature. The released energy maintains the wave while the shock initiates the combustion reaction at the rear.

2.3.2.2 BLEVE
A Boiling Liquid Expanding Vapour Explosion (BLEVE) can be defined as an explosion that results from the failure of a vessel that contains a liquid under pressure with a temperature that is considerably higher than the boiling point under atmospheric conditions. The contents of the vessel before failure consist of a liquid with a satisfied vapor cloud on top. Liquefied Petroleum Gas, (LPG) and CO_2 are examples of gasses that are potentially dangerous in this respect.

Cold BLEVE
In case rupture of the vessel occurs, as a result of an accident for example, the liquid inside starts to vaporize instantaneously due to the sudden decompression. The volume increases tremendously, resulting in very high pressures. Due to boiling of the liquid, this pressure will be sustained until the temperature of the liquid is equal to the boiling temperature under atmospheric conditions.

Hot BLEVE
In case a vessel is exposed to fire, the liquid starts vaporizing at a certain temperature in spite of the increased pressure. Most modern tankers are equipped with a valve that release gas if a certain pressure develops, however this is not obligatory. Besides this, as a result of long exposure to fire, the temperature of the tank will increase and its mechanical properties will change, finally leading to failure. The pressure will drop instantly and the boiling point of the remaining liquid decreases. A large volume of vapor is released and ignites, leading to severe damage in the surroundings of the tank.

Effects
It is very likely that the vessel breaks down as a result of the BLEVE and fragments will be launched driven by the enormous pressure. Besides this, usually a severe fire occurs. The probability of decease for people inside the tunnel in the event of a BLEVE is assumed to be 1 in the Netherlands. For additional information with respect to the effect of explosions on people, reference is made to [9]. The extreme pressure that occurs may result in seriously damage or even collapse of the structure. The probability of occurrence for this phenomenon is between that of a fire and a gas explosion.

2.3.3 Consequences
As stated before, the considered undesired events may cause both material and immaterial damage. Injuries or even casualties are the most striking consequences connected to the events and these are hard to accept by society. Material damage can be severe in an economical sense. If a tunnel cannot be used for a long time as a result of the caused damage, an important link in the infrastructure is suddenly missing.

This will affect many road users and companies that are dependent on transport. In the worst case the entire tunnel is lost and should be rebuilt, resulting in high costs. Besides this, it will take years to recover the connection.

2.3.4 Measures

From analysis, it can be concluded that it is desired to reduce the risk that is connected to a certain event. Depending on the probability of occurrence and the consequences of the event, preventive measures can be taken in order to reduce the risk. These measures may focus on the reduction of the loads or on improving the structural performance of the tunnel. These principles are discussed in more detail, with some examples in the following paragraphs.

2.3.4.1 Limit the probability of occurrence

Since roadaccidents are often the cause of a fire or explosion, some measures concerning the traffic will effectively reduce the probability of occurrence. Speed limitations in tunnels lead for example to fewer accidents. Another effective measure may be passing restrictions for trucks. Strict regulations and requirements concerning the safety and quality of the vehicles are also of importance. The number of vehicles that transport dangerous goods is limited, a suitable solution may be to increase the safety of the vehicles itself. An additional advantage is that safe trucks can pass existing tunnels as well. Furthermore, scheduled allowance of dangerous goods, for instance only during the night might reduce the probability of an accident since fewer other road users are present.

2.3.4.2 Limit the effects

In case somehow a fire or explosion occurs, it is important that the consequences will be limited. Limitations can be achieved by measures that apply for the vehicles or the tunnel structure. These will be discussed in the following.

Vehicles

The most radical measure is obviously the restriction for transport of dangerous goods. In order to make transport possible, vehicles can be designed to limit effects. Trucks that transport liquefied gasses can be equipped with an overpressure valve for example, though this is not obligatory nowadays. The thermo mechanical properties of the material from which a tank is made are also of importance. If a fire will lead to an explosion as well as the magnitude is strongly dependent of these characteristics. Recently research is performed to the application of special coatings that prevent instantaneous failure and increase the resistance against fire for fuel trucks. In this way also the number of casualties as a result of an incident will be reduced. Since the number of trucks that transport goods that are relevant in this respect is only limited, adaptation of these vehicles may be a suitable solution that can be rather easily achieved. Another advantage of this approach is that the trucks can also pass existing tunnels safely.

Design of the tunnel

Furthermore, the tunnel can be designed to limit the effects. It is for example possible to apply fire-resistant coating inside the tunnel in order to prevent propagation of fire. Applying sprinklers, which can fight a fire in an early stage, may be a solution. Roads inside the tunnel are constructed under a slope, in case of leakage of fluid this will be discharged to the sewer quickly. Besides this, very open asphalt concrete will be avoided because flammable fluids can be absorbed in this material. Furthermore, speed limitations will generally lead to less severe accidents. Clearly indicated and good accessible escape routes increase the chance for surviving a fire. Scheduled allowance of dangerous goods may also limit the effects, since relatively few other users are in the tunnel. For explosions, the probabilities for survival are rather low. Limiting the effects by means of structural measures can only be achieved by preventing severe damage or the collapse of the structure.

2.3.5 Conclusion

Which measures concerning safety exactly should be taken is strongly dependent on the particular situation and circumstances. A decision can be based on a risk analysis. It should be noted that in the Netherlands a number of standard measures should be applied in every case. Some measures, like separation of the regular traffic and the transport of dangerous goods, may result in considerable reduction of the risk, though the costs will also increase tremendously. Since it is very unlikely for people inside a tunnel to survive an explosion, the structural measures especially result in limitation of the effects concern the preservation of the structure.

If there is a situation in which this is desired, the design has to be made in such a way that the extreme loads that may occur can be withstood. In case of an explosion, it should in that case be reasonably possible to repair the structure.

2.4 *The Oosterweel tunnel*

This research was triggered by an extraordinary requirement concerning explosion loads in a recent project. The background of this project will be discussed briefly in order to put things in perspective.

2.4.1 Project description

Antwerp is the economical heart of Belgium, as well as an important entrance for the rest of Europe. The city has approximately 500,000 inhabitants, while 1 million people live in its region. Antwerp is of major importance since it accommodates the fourth largest port in the world, which is the second largest in Europe. Besides this, the second largest petrochemical industry in the world is located here.

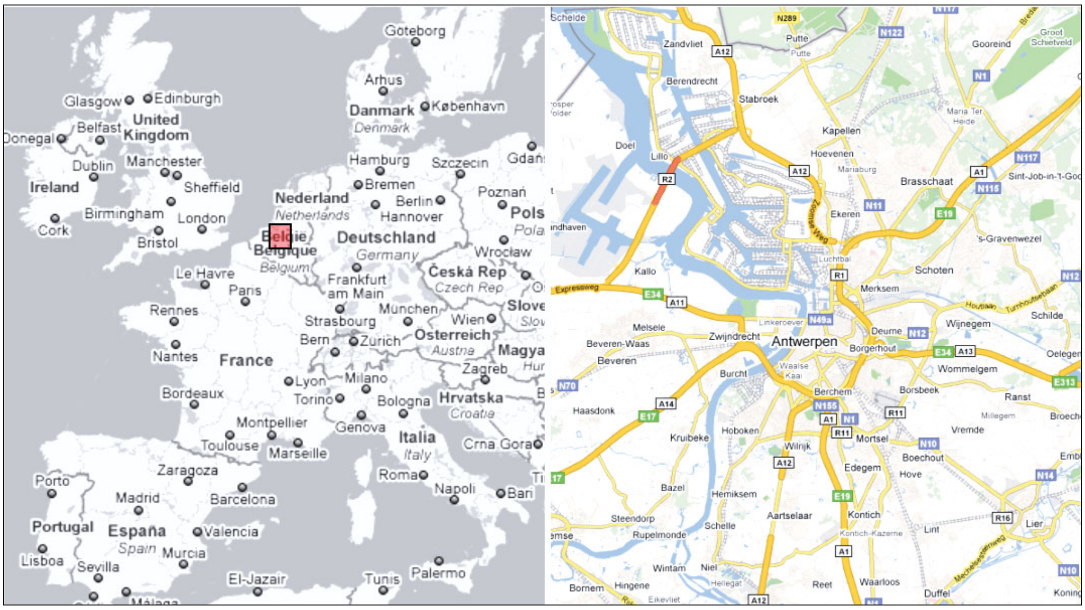


Figure 2-7 Location Antwerp [29]

The economical activities that take place in this highly developed area require the presence of a suitable infrastructural network. In the current situation the capacity is however insufficient. Approximately 186,000 vehicles are daily present on the main roads in the area, of which 25% are trucks. Besides this, predictions for the near future show an increase of transported goods of 25% between 2001 and 2010 and an increase of 40% between 2001 and 2020. This growth can be partly explained by the fact that the port of Antwerp is expanding. In this respect, the Fleming government initiated a study to the situation, back in 1995. This study led in 1998 to the development of a master plan, in order to improve the mobility, livability and safety of the area. The capacity of the road network will be extensively increased by means of new connections and the improvement of existing ones. The plan furthermore consists of several projects that concern the improvement of public transport connections by expanding existing tramlines. Besides this, navigation routes are upgraded by means of increasing the vertical clearance of several bridges and renovation of ship locks. After a feasibility study to several variants was performed, the master plan was approved in 2000. Furthermore, the Beheersmaatschappij Antwerpen Mobiel (BAM), an organization under supervision of the government that is responsible for the coordination, financing and administration of the execution of the master plan, was established. The total costs involved with execution of the several projects are estimated to be in the order of € 3.8 billion.

In figure 2-8, the master plan for Antwerp is schematically depicted.

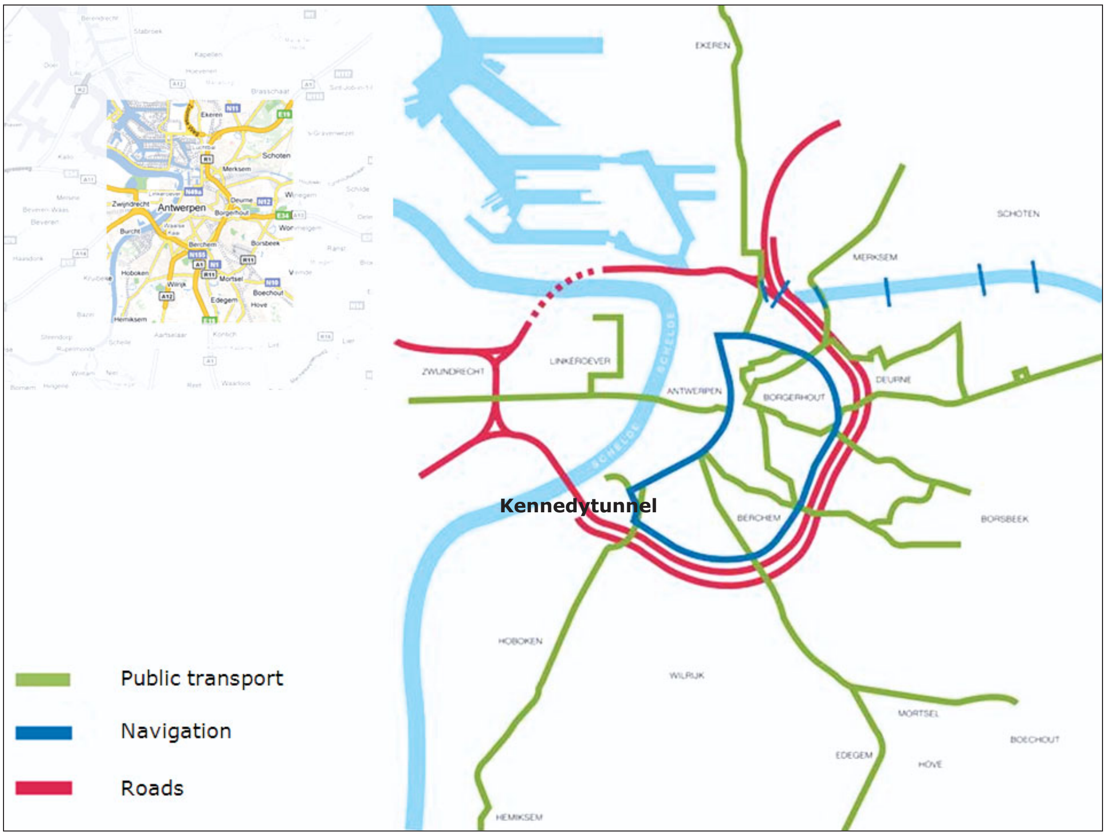


Figure 2-8 Schematic presentation of the master plan Antwerp edited from [29], [30]

One of the projects within this master plan is the Oosterweel connection. Due to the large amount of vehicles that make use of the ring road of Antwerp, congestion is an important problem for the city. Besides this, the safety level on the roads decreases, leading to more accidents. The objective is to close the ring road of Antwerp, in order to increase the capacity and promote the flow. The connection will be partly created by means of a bridge, the 'Lange Wapper' and partly by a tunnel under the Scheldt. The length of the connection is approximately 10 km. The Oosterweel tunnel connects the Oosterweel node, which is the current crossing of the Oosterweelsteenweg and the Scheldelaan, on the right bank with the interchange R1-E17–N49/E34 of which the realization is also part of the master plan at the left bank. The tunnel part is indicated with a dashed line in figure 2-9.

1. Improvement of the capacity of the node R1-E17.
2. Closing the Kennedytunnel for goods traffic.
3. New Urban ring road for local traffic
4. Toll gates at the left bank
5. Scheldt tunnel
6. New node for traffic with destined for the port area
7. Lange Wapper bridge, 4 lanes in each direction
8. Improvement of connections in Deurne

Figure 2-9 Oosterweel connection

The Kennedy tunnel, see figure 2-8, was built in 1969 and designed for 65,000 vehicles per day, in the present situation however, daily 140,000 vehicles make use of this crossing. Approximately 30% of this traffic consists of trucks for which the relative steep slopes of the tunnel are an important disadvantage. As soon as the Oosterweel connection will be completed, the existing Kennedy tunnel, indicated in figure 2-8 will be closed for trucks, besides for the Scheldt tunnel toll will be imposed. The purpose of these measures is to concentrate transport of goods at the new Oosterweel connection while the remaining traffic is encouraged to make use of the old Kennedy tunnel since no toll has to be paid there. As a result, the flow will improve and the degree of safety for the road users will increase.

The estimated costs for the Oosterweel connection are €1.85 billion which makes this a very important project within the entire master plan. In figure 2-9 the different measures involved with this project are schematically presented.

As stated before, the Oosterweel connection partly consists of a tunnel. The client decided that in this case a concrete immersed tunnel should be made. From the traffic prognoses it became clear that 3 lanes in each direction are required, besides a safety lane should be present. Furthermore, a separate tube for cyclists and pedestrians is required, which should be accessible for an ambulance in case of an emergency. Combining a road traffic tunnel and a cyclist tunnel is not very common, only in very specific cases there is a need to do so. Therefore, the cross-section will differ compared to most existing immersed tunnels and is not typical.

2.4.2 Requirement concerning explosions

A remarkable requirement for this project is that an explosion load schematized by a statically distributed load of 500 kN/m^2 directed outwards of the tube and 300 KN/m^2 directed inwards of the tube should be taken into account. A pressure of 500 kN/m² corresponds to a pressure of 50 meters water column. Usually, an immersed tunnel is constructed approximately 20 meters below the water table. Therefore, the order of magnitude of the explosion load that should be taken into account is severe.

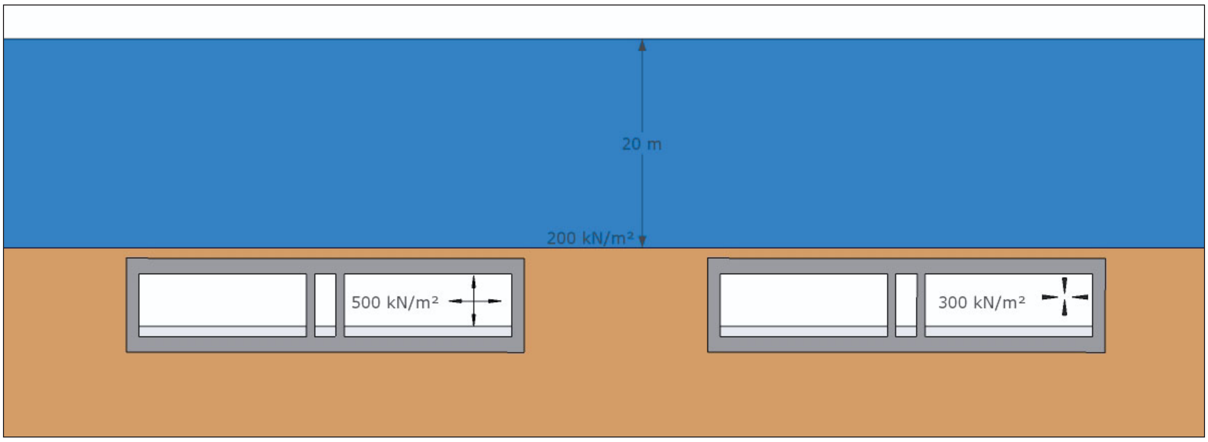


Figure 2-10 Explosion schematized as a combination of a static pressure (left) and suction (right)

BAM Infraconsult participated in the tender phase. During the preliminary design, it became clear that the explosion load that should be taken into account results in relatively large amounts of reinforcement and excessive dimensions. In order to investigate the nature and relevance of this requirement as well as the need for an exploring study to possible solutions, this thesis is initiated.

3 Regulations concerning structural safety in tunnels

For the undesired events that may occur as a result of an accident in which dangerous goods are involved, regulations with respect to the design of a tunnel apply. These will be discussed in the following paragraphs, distinction is made between the occurrence of fire, gas explosions and BLEVE.

3.1 *Fire*

In case of a fire, the temperature inside the tunnel may rise very high. This may result in reduction of the structural capacity. Two mechanisms are important in this respect.

- In case the structure is exposed to extreme heat, the tensile strength of the reinforcement steel decreases. This may results in large deformations or even collapse.
- The moisture that is present inside the concrete expands due to the temperature rise and a pressure develops inside the material. This may lead to spalling of the cover layer, whereas the reinforcement will be uncovered and therefore will be directly exposed to the fire.

According to the Eurocode, a tunnel should be designed in such a way that the structural integrity during and after a normative fire is ensured.

In order to comply with this requirement, there are two major posibilities.

- Requirements concerning the dimensions of the structural elements
- Application of protective layers

Rijkswaterstaat, the Dutch ministery of transport defined a normative fire load to be applied for tunnels. This load is based on a fire due to a petrol tanker with a volume of 50 m^3. The fire load is 300 MW, and the fire lasts for 120 minutes. The temperature as a function of the time is presented in figure 3-1.

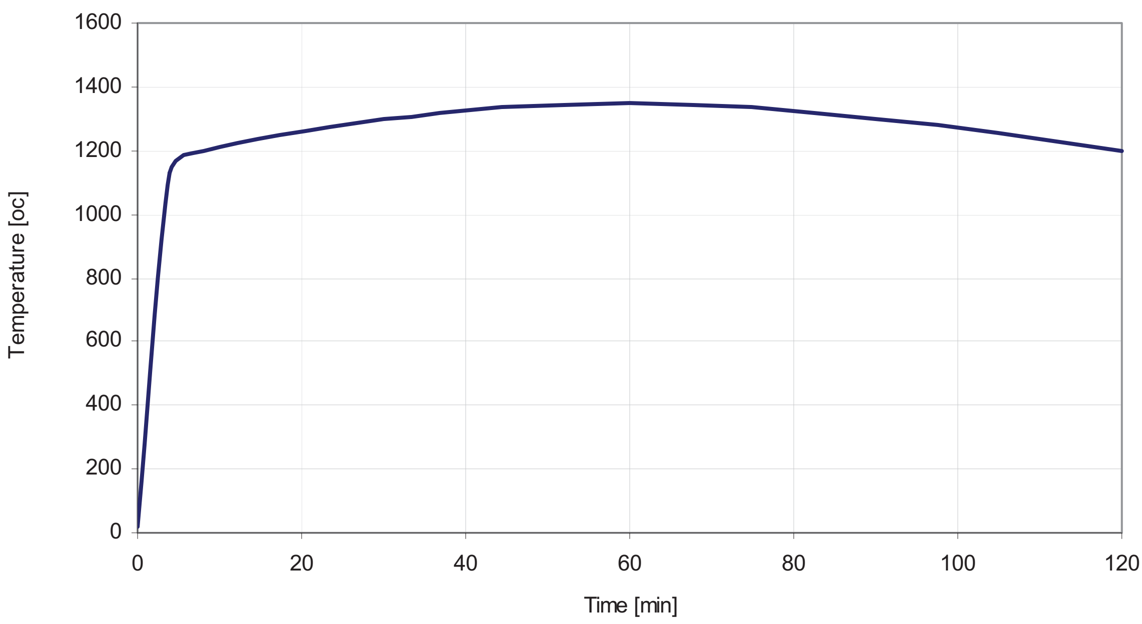


Figure 3-1 Normative fire load according to Rijkswaterstaat

It is required that the temperature of the reinforcement steel should not exceed a temperature of 250 ^{0}C whereas the temperature of the interface between a protective layer and the reinforcement should not exceed 380 ^{0}C.

3.2 *Gas explosions*

In this paragraph it will be explained how to account for gas explosions according to the current regulations. Furthermore, the results of research performed by TNO will be discussed, in order to give a more complete survey.

3.2.1 Gas explosion according to Eurocode

In NEN-EN 1991-1-7 (Eurocode), [6], the following is stated with respect to explosions.

'Explosions shall be taken into account in the design of all parts of the building and other civil engineering works where gas is burned or regulated, or where explosive material such as explosive gases, or liquids forming explosive vapour or gas is stored or transported (e.g. chemical facilities, vessels, bunkers, sewage constructions, dwellings with gas installations, energy ducts, road and rail tunnels).'

The informative annex D, of [6], can be used to comply with this statement in a deterministic way. Since it is an informative annex it is not compulsory to be used. Alternatively, informative annex B of the same code provides a method based on risk analysis. Both principles will be discussed in the following paragraphs.

3.2.1.1 Deterministic approach

In annex D of [6] a number of relations between the pressure as a result of a gas explosion as a function of the time and distance are given. These formulae are generally evaluated within the framework of this research. The Eurocode distinguishes different approaches for a detonation and a deflagration, these will be discussed in the following paragraphs.

Detonation

In case of a detonation, the following expressions should be used.

$$p(x,t) = p_0 \exp\left\{-\left(t - \frac{|x|}{c_1}\right)/t_0\right\} \; for \; \frac{|x|}{c_1} \le t \le \frac{|x|}{c_2} - \frac{|x|}{c_1} \tag{3.1}$$

$$p(x,t) = p_0 \exp\left\{-\left(\frac{|x|}{c_2} - 2\frac{|x|}{c_1}\right)/t_0\right\} \; for \; \frac{|x|}{c_2} - \frac{|x|}{c_1} \le t \le \frac{|x|}{c_2} \tag{3.2}$$

$$p(x,t) = 0 \; for\ all\ other\ conditions \tag{3.3}$$

Where:

p_0 = the peak pressure $(= 2000\,\text{kN/m}^2$ for a typical liquefied natural gas fuel$)$

C_1 = the propagation velocity of the shockwave $(\sim 1800\,\text{m/s})$

C_2 = the acoustic propagation velocity in hot gasses $(\sim 800\,\text{m/s})$

t_0 = the time constant (= 0.01 s)

$|x|$ = the distance to the heart of the explosion in [m]

t = the time [s]

With help of the software package MAPLE, these expressions were considered in more detail. In figure 3-2 the pressure as a function of the time and distance to the centre of the explosion is depicted. The irregular character of the pressure wave is caused by the limited number of grid points that is used for the generation of this figure. Obviously, the peak pressure has a constant value of 2000 kN/m^2.

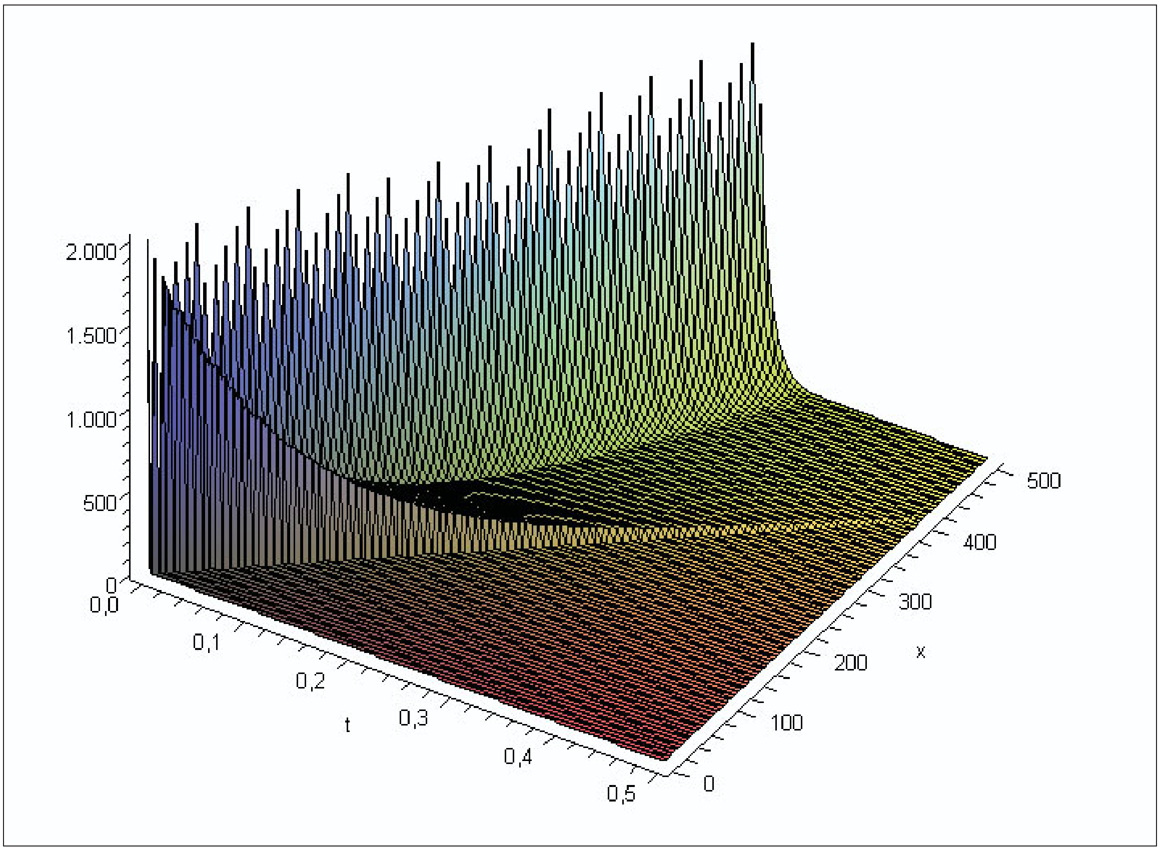


Figure 3-2 Pressure as a result of a detonation as a function of time and distance

For a number of distances from the centre of the explosion, the pressure development as a function of the time was considered, as indicated in figure 3-3. In fact, these figures can be considered to be cross-sections of figure 3-2.

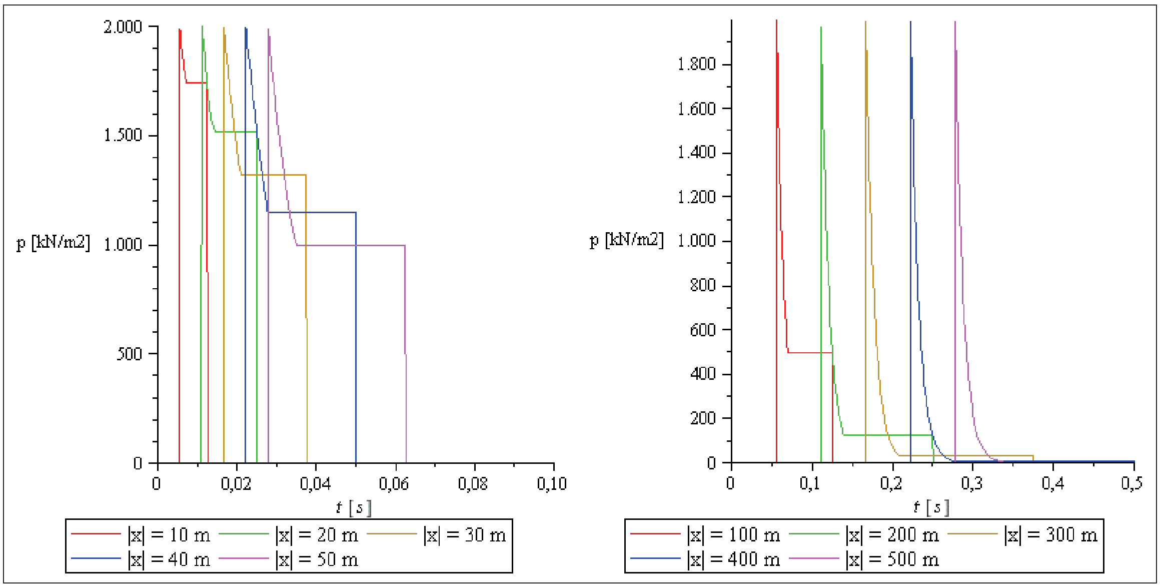


Figure 3-3 Pressure development in time for several distances

According to the relations, the peak pressure, p_0 should be taken into account no matter the distance from the center of the explosion. The shockwave propagates through the tunnel tube while the peak pressure will not decrease. Since a tunnel is an isolated environment, this seems a logical schematization, though it is somewhat conservative. As can be seen from figure 3-3, the area under the curve seems to increase for the first 50 m. Physically, this area correspond to the impulse at a certain distance.

$$I(x) = \int_{\frac{|x|}{c_1}}^{\frac{|x|}{c_2}} p(x,t)\,dt \qquad (3.4)$$

For several distances the surface under the curve is calculated by means of integration, the result is depicted in figure 3-4.

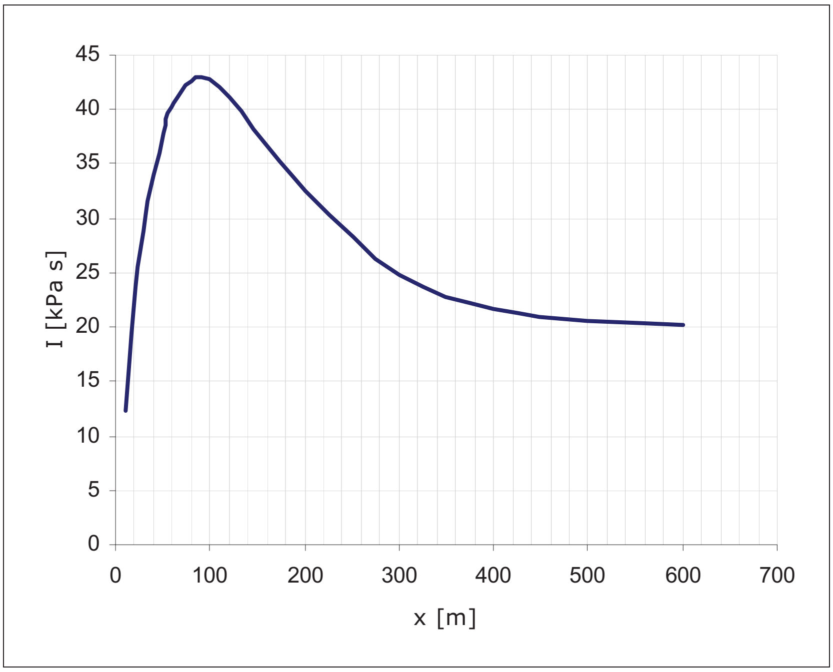


Figure 3-4 Impulse as a function of distance

The impulse seems to be increasing with the distance for the first 100 m. This is strange, since the impulse in a cross-section at a certain distance from the centre of the explosion should not exceed the amount of impulse in the centre of the explosion. Considering results of tests by TNO which will be discussed in 3.2.2, it could be assumed that the gas cloud will be ignited from the edge. The definition of the parameter x, distance to heart of the explosion implies ignition in the centre however. It is not clear how to deal with this observation, in the code there are no restrictions or limitations of the method stated that provide an explanation. It is also unclear which pressure development is normative. Since the peak value is equal for all distances, the question remains what is worse for the structure, a very short high pressure or a longer, somewhat lower pressure. The code provides no information or explanation with respect to this aspect and the applicability of the described method is not clear, this is confirmed by [32]. Therefore, within the framework of this research it will not be considered further.

Deflagration
In case of a deflagration, the following expression for the pressure development as a function of the time should be used according to [6].

$$p(t) = 4p_0 \frac{t}{t_0}(1 - \frac{t}{t_0}) \text{ for } 0 \le t \le t_0 \tag{3.5}$$

Where:

p_0 = the peak pressure (= 100 kN/m^2 for a typical liquefied natural gas fuel)

t_0 = the time constant (= 0.1 s)

t = the time [s]

The pressure may be used for the entire interior surface of the tunnel. In figure 3-5 the graph of the expression is depicted.

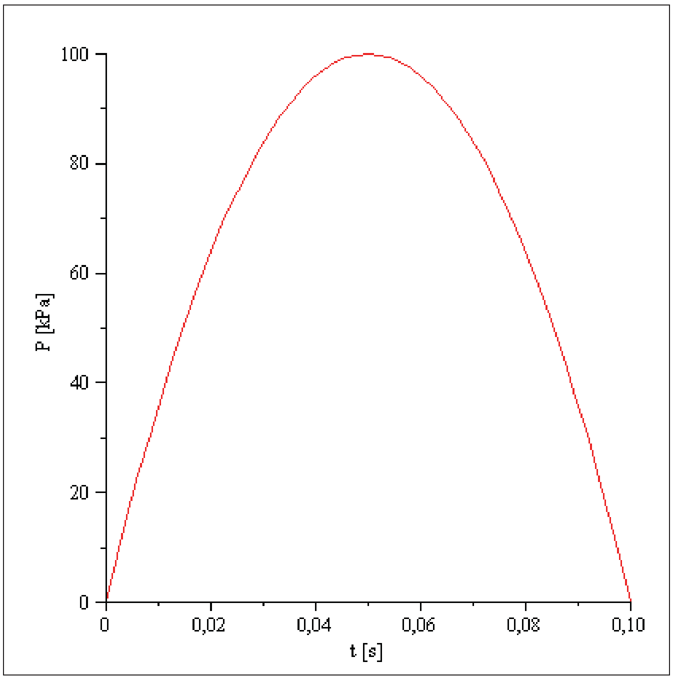


Figure 3-5 Pressure development for a deflagration

This requirement is usually taken into account in the design of an immersed tunnel. The peak pressure of 100 kPa is taken into account as an equally distributed static load. For the magnitude of this requirement a practical value is determined in such a manner that it can be easily withstood by common cross-sections of immersed tunnels. Therefore, it has hardly anything to do with designing for explosions.

3.2.1.2 Risk analysis

As stated before, the classification of a tunnel and the nature and extent of preventive measures that should be taken can also follow from of a risk analysis. Many aspects are involved in this consideration. The schematic representation of this process according to annex B of [6] is depicted in figure 3-6. In the following paragraphs, this scheme will be explained in more detail aggravated to road traffic tunnels.

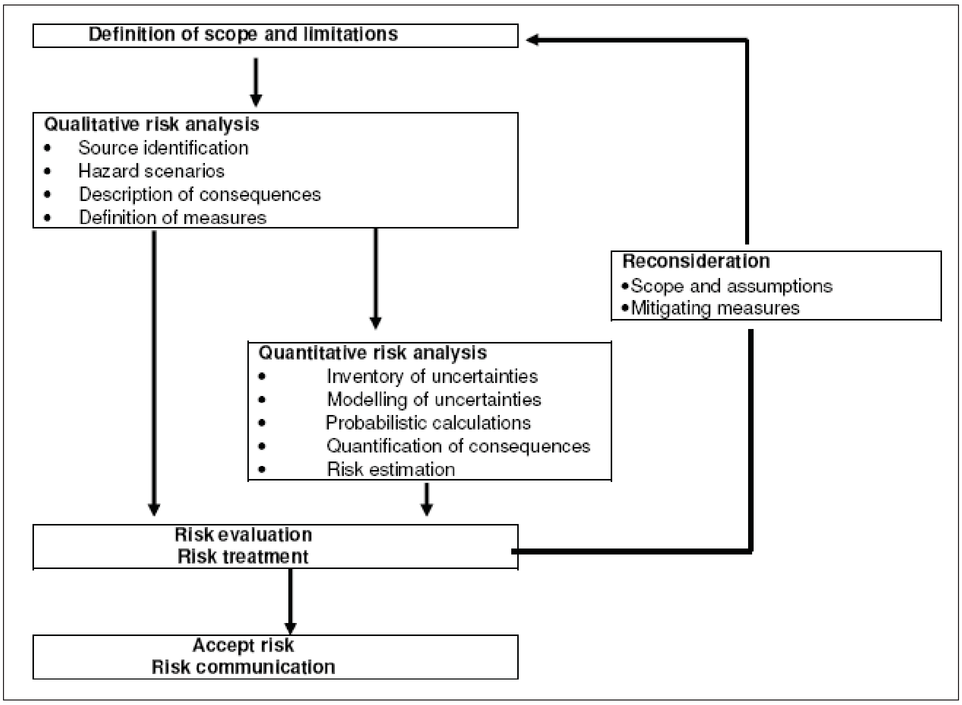


Figure 3-6 Flow chart risk analysis [6]

Definition of scope and limitations

This stage concerns the description of the situation, background and objectives of the risk analysis. Furthermore, all technical, organizational, environmental and human circumstances should be stated. Besides this, all simplifications, assumptions and starting points should be made clear. The result of this stage is highly dependent on the particular case that is considered.

Qualitative risk analysis

The risks involved with the transport of dangerous goods through a tunnel are closely related to the effect that these goods may have in case of undesired event. Such an event can have several causes, for example a traffic accident, both as a result of human failure or failure of a vehicle, an earthquake, terrorist attack or failure of the tunnel structure. Furthermore, a combination of the mentioned causes may lead to an undesired event. The classification of the dangerous goods and the possible effects are already presented in paragraph 2.2.1. It is important to have an idea of all possible undesired events that may occur. For tunnels, a number of normative scenarios are defined. These serve as a guideline for the determination of the extreme loads that may act on the structure. The consequences of an undesired event should also be clear. For each hazard scenario, the expected damage and number of casualties as well as the nature of injuries are determined. Internal consequences concern the tunnel itself, while external consequences cover effects on the surroundings. External consequences may affect a large area and many people besides the direct users of the tunnel. Based on the foregoing aspects, a number of possible measures can be taken, both to prevent the occurrence of an undesired event and to limit the consequences.

Quantitative risk analysis

The result of the qualitative risk analysis is in fact a description of the situation. In order to make decisions concerning the design and safety measures, the analysis should be made in a quantitative manner. For the occurrence of undesired events, probabilities have to be estimated. Furthermore, the consequences should be considered quantitatively. This can be done based on available data for example. The entire situation can be modeled and probabilistic calculations can be performed. In this way, a rather reliable estimation of the risks involved can be made. In the Netherlands, standards to comply with were defined based on research to a large number of locations. These standards make a distinction between the individual risk and the group risk. The individual risk is defined as the probability for a lethal situation at a certain distance of the considered activity. Obviously, the risk decreases with an increasing distance to the source, which can be presented with a contour map. It was determined that the individual risk should be lower than 10^{-6} per year. The group risk is defined as the probability of a disaster with a certain number of fatalities. For the group risk, it was stated that this should be lower than 10^{-4} $km^{-1}year^{-1}$ in case of 10 fatalities, 10^{-6} km^{-1} $year^{-1}$ in case of 100 fatalities and so on [19].

Evaluation/reconsideration
Based on the analysis as described in the foregoing paragraphs, it should be decided if the risks involved are acceptable. If this is not the case, adaptations in the scope and assumptions are required or supplementary measures should be taken. Again, a risk analysis should be performed to check if the changes have the desired effect. This process continuous until an acceptable situation is created.

Practice
Due to the relatively low probability of occurrence, generally from the risk analysis it follows that the risk of collapse due to an explosion should be accepted. There may however be situations in which the risk of an explosion is rather high, as a result of a high probability or severe consequences. In such a situation it may be desirable to take structural measures in order to limit the economical loss. This may concern a busy connection where relatively intense transport of explosive goods is present for instance. If it is concluded that structural measures are necessary, the code provides only the deterministic approach, discussed in paragraph 3.2.1.1

3.2.2 Gas explosion according to TNO

From research by TNO, [11], a relation between the distance and the occurring pressure and impulse as a result of a gas cloud explosion was determined. The transition from a deflagration to a detonation in tunnel tubes was experimentally determined to vary between a flame propagation of 600 and 1000 m/s, a value of 800 m/s is used in the model. It is assumed that the propagation of the flames increases linear to the transition. For the constant speed of the detonation wave, the following relation holds.

$$S_{CJ} = \sqrt{\frac{2 \cdot Q \cdot (\gamma_1^2 - 1)}{c_0^2}} \tag{3.6}$$

Where:

S_{CJ}	= Speed of the detonation wave	[m/s]
Q	= Combustion heat of the mixture	[J/kg]
γ_1	= Ratio specific heat	[–]
c_0	= Acoustic propagation speed in atmosphere	[m/s]

The distance at which the transition takes place, R_{det} is experimentally determined for several situations. In figure 3-7 the relation between the speed and distance is depicted.

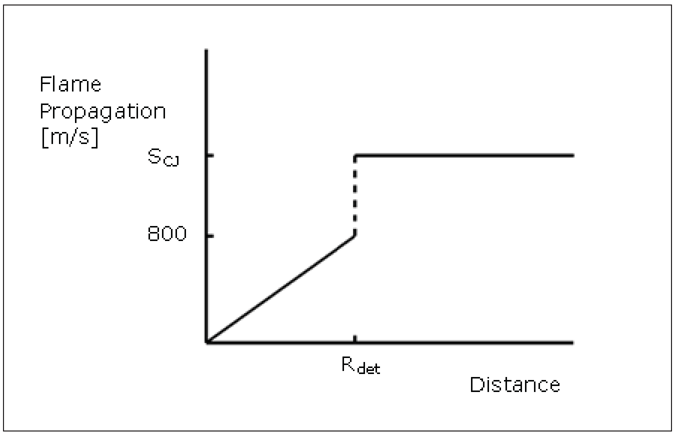


Figure 3-7 Transition between deflagration and detonation [7]

A gas explosion can be modelled as depicted in figure 3-8, where t_d is the duration of the pressure, β is the ratio of the time during which the pressure is increasing and the total duration of the pressure.

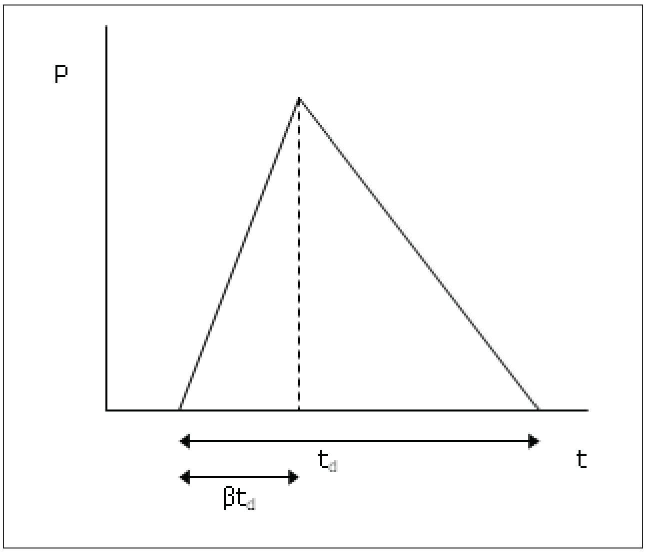


Figure 3-8 Model for gas explosion

Two parameters were considered to be of major importance, these are the length of the gas cloud and the location of ignition. Furthermore, it is assumed that the cloud is situated in the center of the tunnel, the length of the tunnel is 1000 m and a traffic jam is present. For a propane air mixture, the results for a central ignited gas cloud are depicted in figure 3-9 for several lengths. Besides this, a number of general conclusions are listed. These figures can be used a rules of thumb.

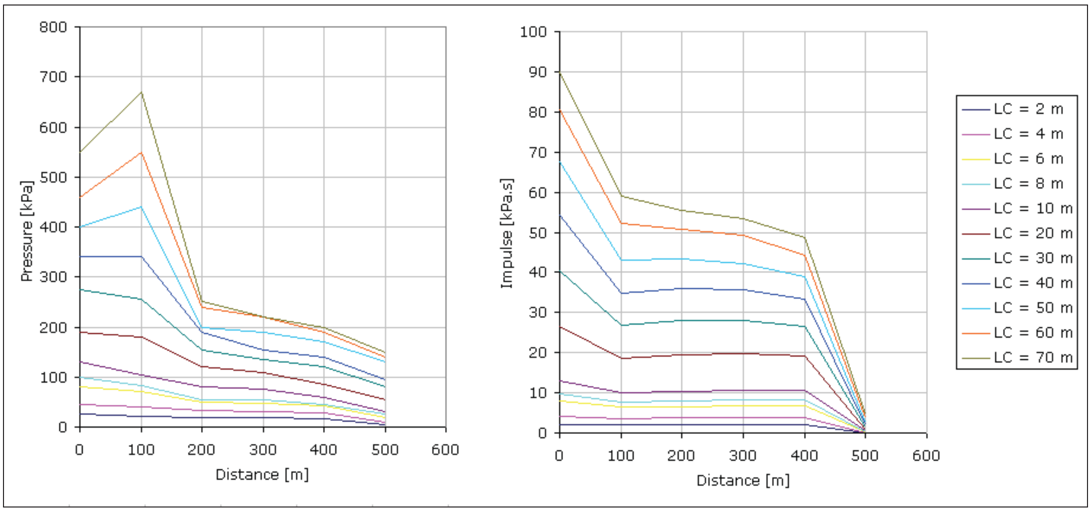


Figure 3-9 Pressure and momentum versus distance for ignition in the center of a propane gas cloud

- The overpressure increases approximately proportional with the length of the cloud.
- The maximum pressure occurs in the ignition point.
- For higher pressures, stronger the decrease with respect to the distance are found.
- Near the entrance, the pressure drops significantly.
- The momentum decreases fast initially, remains constant inside the tube and decreases fast near the entrance.

For gas clouds that are ignited at the edge the results are presented in figure 3-10. For longer gas clouds, no reliable data for the impulse was found.

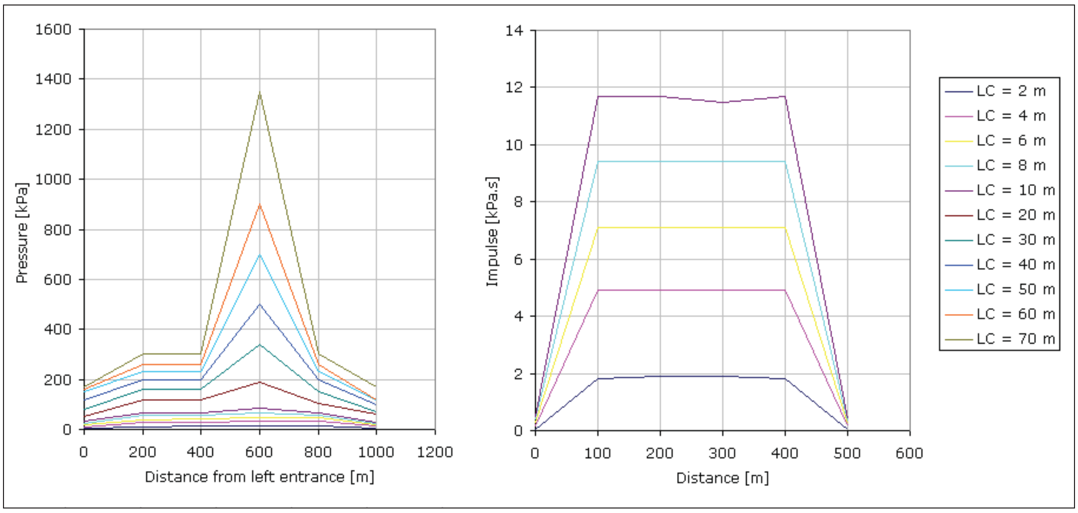


Figure 3-10 Pressure and momentum versus distance for ignition at edge of a propane gas cloud

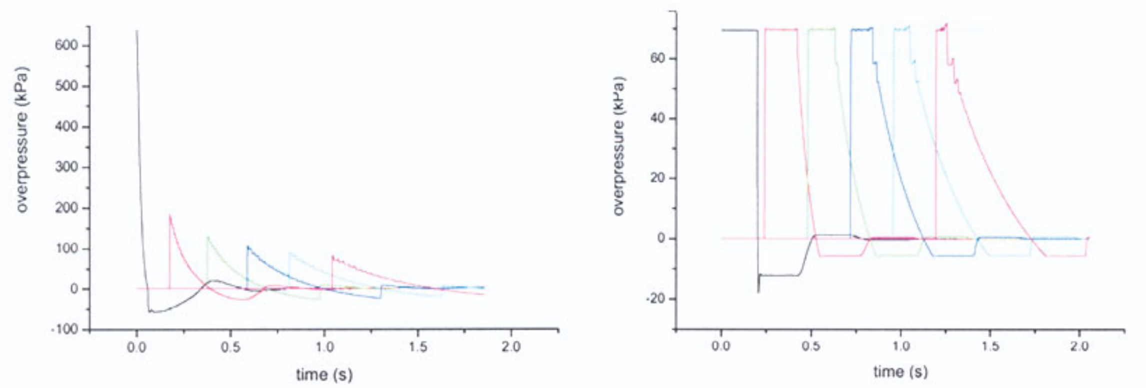
- The overpressure increases approximately proportional with the length of the cloud.
- The maximum pressure occurs at the point where the propagation speed of the flames is maximal.
- The momentum increases first, remains constant and decreases very fast near the entrance.

Practice

It can be concluded that the length of the developed gas cloud is of major importance to the magnitude of the occurring pressure and impulse. It is however very unlikely that a large gas cloud will develop inside a tunnel. In case liquid leaks from a damaged vessel a pool will develop, depending on the nature of the liquid, subsequently vaporizing may occur, an example is gasoline. In order to prevent the development of a pool, usually the roads inside a tunnel are built under a slope and the fluid will be discharged through explosion safe sewers and pumps. Apart from that, there are usually a lot of sources for ignition present inside the tunnel, like engines of the vehicles and hot exhausts for instance. In case a gas cloud will develop, it will be probably ignited soon resulting, in a fire. The occurring pressures will however be limit. It can be concluded that gas explosions are not of importance for tunnels, which is supported by [15].

3.3 *BLEVE according to TNO*

The probability of occurrence of a BLEVE is estimated to be in between those of a fire and a gas cloud explosion. Furthermore, the consequences can be severe. Nevertheless, there are no regulations concerning additional loads due to this phenomenon. There is however research concerning BLEVE's performed. In a report concerning tunnel safety by TNO, [16], a model for the occurring pressure as result of a BLEVE is presented. It is stated that a conservative approach is appropriate, because many aspects are unclear jet. A numerical model was made, based on the Euler equations. A very important factor for the pressure development inside a tunnel is the release time. The faster the release is, the higher the occurring overpressure will be. The natural upper limit is the vapor pressure. An example from [16] is depicted in figure 3-11, it can be concluded that the differences in the occurring pressures for the two cases are very large. It can be concluded that it could be very effective to design the trucks that transport goods that are dangerous with respect to the occurrence of a BLEVE in such a way that instant failure is impossible. In this way, the load will be reduced significantly.



Instantaneous release of 50 m³ propane in tunnel of 60 m² cross-section blast at 0,100,200,300,400 and 500 m distance

0.2 s release of 50 m³ propane in tunnel of 60 m² cross-section blast at 0,100,200,300,400 and 500 m distance

Figure 3-11 Influence of the release time on the occurring pressure [16]

For the calculation of the pressure wave, two applicable methods are described by TNO. At first, the Bakers method can be used, the following relations hold [7].

$$E_{\text{explosion}} = \frac{(P_{init} - P_{atm}) \cdot V}{\gamma - 1} \tag{3.7}$$

Where:

V = The volume of the gas

γ = The ratio of the specific heats of the gas

The occurring peak overpressure, p_s, at a certain distance can be determined with the following relations below and figure 3-12.

$$\bar{R} = r \left(\frac{P_{atm}}{E_{explosion}} \right)^3 \tag{3.8}$$

$$\bar{P}_s = \frac{P_s}{P_{atm}} - 1 \tag{3.9}$$

Where:

$\bar{P}$	=	The scaled peak overpressure	[kPA]
r	=	The distance	[m]
$\bar{R}$	=	The scaled distance	[m]

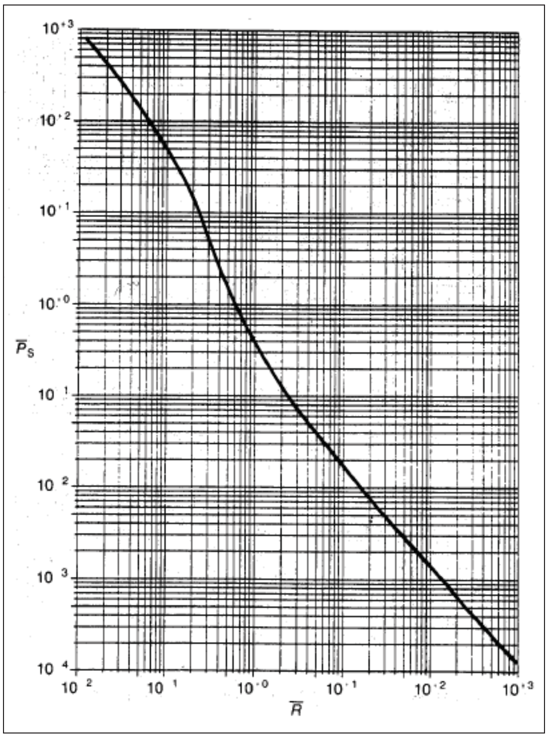


Figure 3-12 Scaled peak overpressure vs. distance [7]

Secondly, according to [11], the shock tube formula can be used. The following relation is valid.

$$\frac{p_1}{P_a} = (\bar{P}_{so} + 1)\left[1 - \frac{(\gamma_1 - 1)\left(\frac{a_a}{a_1}\right)\bar{P}_{so}}{\left[2\gamma_a\left\{2\gamma_a + (\gamma_a + 1)\right\}\bar{P}_{so}\right]^{\frac{1}{2}}}\right]^{-2\gamma_1/(\gamma_1 - 1)} \tag{3.10}$$

Where:

P_1	=	Initial pressure in the vessel	[kPa]
P_a	=	Atmospheric pressure	[kPa]
$\bar{P}_{so}$	=	Initial dimensionless blast pressure	[-]
γ_a	=	Ratio specific heats of air	[-]
γ_1	=	Ratio specific heats of the compressed gas	[-]
a_a	=	Acoustic propagation speed in air	[m/s]
a_1	=	Acoustic propagation speed in the gas	[m/s]

In the vicinity of the BLEVE, the tunnel is exposed to the initial overpressure that acts perpendicular to the walls and roof. The pressure will be reflected resulting in a reinforced pressure at the structure. Away from the BLEVE, the reflected pressure is of less importance, and the structure will be exposed to the initial pressure. The magnitude of the reflected pressure can be determined with the following relation.

$$P_r = \frac{8 \cdot P_s^2 + 14 \cdot P_s \cdot 10^5}{P_s + 7 \cdot 10^5} \qquad (3.11)$$

Where

P_s	=	The incoming pressure	[kPa]
P_r	=	The reflected pressure	[kPa]

3.3.1.1 Order of magnitude

For a number of liquids, the blast pressure for different temperatures is listed in tables that can be used as rules of thumb. These can be found in [11]. It should be noted that these numbers are rather indicative. The phenomenon BLEVE turns out to be very complex, research to the exact mechanisms and order of magnitude still continues. For recent studies, it is attempted to determine the representative load for a tunnel as a result of a BLEVE. In consultation with Dr. Ir. J. Weerheijm, TNO, the representative BLEVE load according to the most recent insights is explained. This will be discussed in detail in paragraph 4.5.

3.4 *Terrorism*

Apart from accidental explosions, it may also occur that a tunnel becomes the target of an attack by terrorists. Since the terrorist strike in the USA 2001, also Europe has suffered from attacks. In 2004, trains were blown up in Madrid, Spain resulting in 191 casualties and 1400 wounded. In 2005 several bombs exploded in the London subway, 56 people died in this event whereas 700 where wounded. In the Netherlands, there has been no actual attack recently, though there is a threat which is reinforced by the participation of Dutch forces in military operations in Afghanistan. In 2001, the Coentunnel, Zeeburger tunnel, Botlektunnel and Beneluxtunnel were closed by the police and military as a reaction to an anonymous letter that contained a detailed description of an intended attack.

Since especially during the rush hours a lot of people may be present inside the isolated tubes, this could lead to a disaster. Furthermore, tunnels are of great importance for economical activities. Therefore these structures can be considered as potential targets.
Since terrorists are willing to give their life for the success of their action, it is very hard to prevent the often well planned attacks. Furthermore, the methods that are used are in many cases unexpected and well prepared.

Various scenarios can be thought of for an attack on a tunnel. Explosives may be hidden in a vehicle and detonated at the desired moment. Alternatively explosives may be applied to the structure. Furthermore, a truck containing fuel or similar substance may be detonated on purpose. No matter the method, the difference with an accidental explosion is that an attack is a planned action to cause destruction on purpose. The materials that will be used as well as the location and moment of the explosion can be carefully planned in order to achieve the most striking effect.

There are no requirements concerning terrorism in the design codes that apply for tunnels in the Netherlands and the threat of an attack by terrorists is considered low currently. The possibilities to increase the safety for people inside the tunnel by means of structural measures are very limited. Within the framework of this research, therefore no further attention will be paid to this phenomenon.

3.5 *Conclusions*

For an immersed tunnel, several undesired events can be thought of. Fire and explosion are most striking for the safety of people inside the tunnel as well as for the structural integrity. From literature and available results of recent research, discussed in the foregoing paragraphs, the following conclusions are drawn.

- The regulations that apply for tunnels in the Netherlands provide requirements concerning fire resistance.
- For gas explosions there are only informative codes available which are not compulsory to be used. Both a deterministic approach and a guideline for risk analysis are provided.
- The theoretical background, as well as the applicability of the deterministic approach for gas explosions presented in the Euro Code is not clear and the method gives unexpected results.
- Research to gas explosions by TNO gives logical results and the parameters that are of influence on the magnitude are clearly described.
- It can be concluded that the probability of occurrence of a gas explosion of a dangerous magnitude inside a tunnel can be neglected.
- The phenomenon BLEVE is complex and it is not completely understood at this moment.
- There are some indicative rules of thumb available as well as results of recent research by TNO.
- A BLEVE is considered the normative type of explosion since the probability of occurrence is high compared to a gas explosion, whereas the consequences are probably more severe.
- The event of an explosion due to an attack by terrorists could have severe consequences. Though within the framework of this research it will be not considered since it is not feasible or even achievable to ensure that a tunnel is resistant against an attempt to destroy it.

The conclusions for the considered phenomena are summarized in table 3-1.

Scenario	Eurocode	Research available	Considered to be relevant
Gas cloud explosion	Yes	Yes	No
BLEVE	No	Yes	Yes
Fire	Yes	Yes	Yes
Terorristic attack	No	unknown	No

Table 3-1 Conclusions for the considered scenarios

4 Structural response to an explosion load

In this chapter, the effects of an explosion on the structure will be discussed. A typical cross-section of an immersed tunnel will be considered at first and the weak points are indicated. Subsequently, attention will be paid to the modeling of the response to an explosion load. Different models will be discussed and compared. The results will be used to review and judge the relevance of the static requirement concerning explosion loads that was stated for the Oosterweel tunnel recently.

4.1 *Qualitative consideration*

The structural response of a tunnel to an explosion will be discussed qualitatively first. To this end the flow of forces in a typical cross-section will be evaluated. Subsequently, the resistance against an explosion will be generally discussed.

4.1.1 Typical cross-section of an immersed tunnel

As stated before, the design of an immersed tunnel is strongly influenced by the fact that the elements should be afloat during transport. The required amounts of structural concrete and the additional ballast concrete normally result from optimization. In the final situation, the structure is exposed to several permanent loads. The most important are the water pressure and soil pressure that act on the outer walls and roof, as indicated in figure 4-1. Since no horizontal loads act on the intermediate walls, these can be of relatively small dimensions compared to the other elements. In fact, the intermediate walls are just vertical supports for the roof and floor. Usually high explosion loads are not taken into account for this type of tunnels.

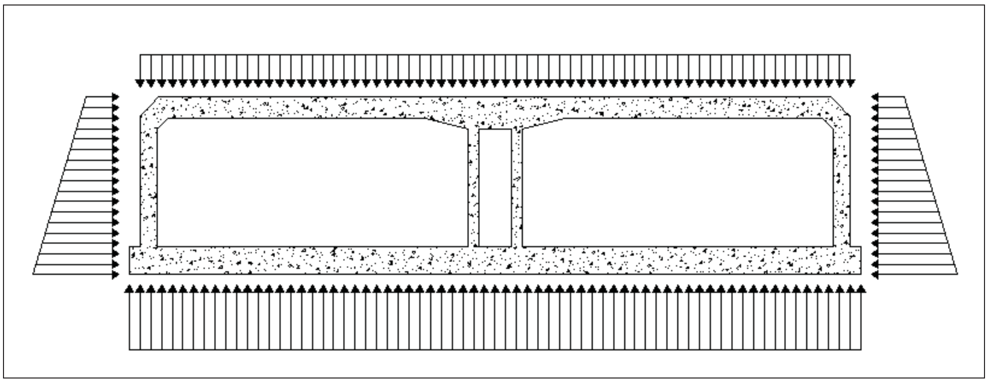


Figure 4-1 Typical cross-section with permanent loads

4.1.2 Resistance to explosion loads

In case of an explosion, an overpressure inside the tunnel occurs, introducing severe forces. As a result of the pressure, all elements are loaded in a different direction and manner compared to regular circumstances. Especially the relatively thin intermediate walls are sensitive for the suddenly occurring large horizontal loads and seem to be the weakest link at first glance. If these intermediate walls fail, the roof is not supported properly anymore and cracks resulting in leakage, large deformations or even collapse are to be expected. The deadweight of the several elements provides resistance against the occurring pressures. Besides this, the permanent loads that are acting on the roof, floor and outer walls have a favourable influence on the resistance. As a result of the high loads that act on the floor, this element can be considered as rather stiff compared to the rest of the structure. For the intermediate walls no distributed load is present, therefore these elements are even more vulnerable. Apart from the intermediate walls, the roof of the tunnel may be a weak link. Since the elements are completely embedded in the subsoil, the outer walls and floor are very well supported. The roof is only covered with a relatively thin layer of sand or rock. In case the explosion is powerful enough, the connections between the walls and roof may fail and the roof can be lifted. The above-mentioned aspects deal with the direct effect of the sudden impact that results from an occurring explosion. The entire structure will experience deformation and displacements that are oriented outwards. Since the structure is only exposed to the explosion load for a very short time, spring back effects may threat the structure as well. The very high pressure that is suddenly present disappears also in an instant. The permanent loads are still present however and become dominant again. It should be noted that there is also interaction between the structure and the surrounding soil.

4.1.3 Estimation of occurring damage

In order to make prognoses of the occurring damage in case of an explosion, TNO developed several damage diagrams that can be used for evaluation of the effects on typical cross-sections [11]. Therefore, a number of representative cross-sections was considered. The focus is on the weakest parts of the cross-section, which are the intermediate walls and roof as stated in paragraph 4.1.2. Distinction was made in the schematization of the structure, the connection between the roof and intermediate walls was considered to be simply supported as well as clamped, as indicated in figure 4-2.

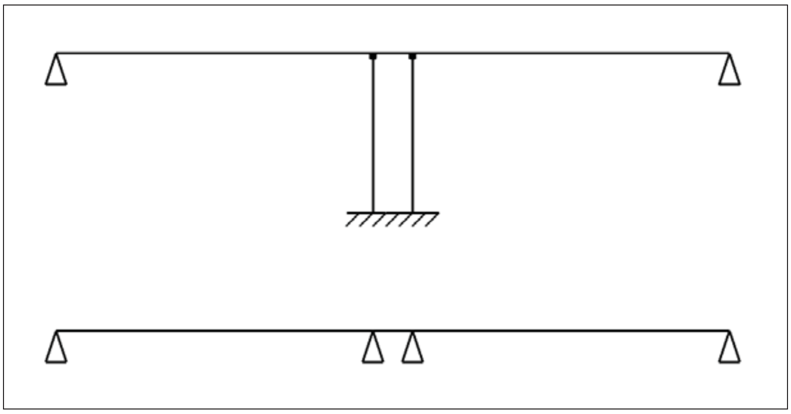


Figure 4-2 Different structural schemes

Three damage levels were defined in order to classify the occurring damage, these are listed below.

- Threshold of cracking of the concrete;
- Threshold of yielding of the reinforcement steel;
- Failure of the clamped connection between the roof and intermediate wall.

Failure of the structure is the result of the development of plastic hinges, leading to a failure mechanism. Therefore, the moment-deformation curves in the locations where a plastic hinge can be expected are of importance.

The third criterion follows from a military manual for the design of explosion resistant structures. Failure is supposed to occur if the rotation at the clamped connection exceeds 2^0. These three criteria were visually presented in diagrams as function of the pressure and impulse. As stated in paragraph 3.2.2, the development in time is very important for the magnitude of the load, therefore the diagrams were made for several durations. Obviously, a clamped connection between the intermediate wall and roof has a positive influence on the resistance of the roof against explosion loads. Response is assumed to be dominated by the first natural mode of the model. From the research, it can be concluded that failure occurs initially at the connection between the roof slab and intermediate wall. The pressure impulse diagrams can be used as a first indication, it should be noted however that these are developed for typical cross-sections, for which no special provisions concerning the resistance against an explosion are applied. Therefore, these figures are especially applicable for reviewing the resistance against explosions of existing tunnels. For designing an explosion resistant tunnel, these are less suitable. Severe explosion loads will probably require a cross-section different from a regular tunnel.

4.2 *Modeling of the structural behavior*

In this paragraph, recent research by TNO to the structural response of tunnels will be discussed briefly. Furthermore, it is attempted to make a simple model in order to estimate the order of magnitude of the response. The developed model is discussed and the results will be compared to those of the research by TNO.

4.2.1 Research into the effect of explosions in tunnels by TNO [20]

In order to estimate the response of a tunnel to an explosion load, TNO in 2007 developed a finite element model. The background and characteristics of this model will be discussed briefly in this paragraph.

Schematisation of the load

Three existing tunnels were considered within the framework of this research, these are the Caland tunnel, the Drecht tunnel and the Leidsche Rijn tunnel. The load case that was considered concerned a BLEVE that is schematised by an exponentially decaying load. Since the geometry of the cross-section is of influence on the load, for each tunnel a characteristic load scenario was determined. The load case that TNO used for the evaluation of the Caland tunnel is presented in figure 4-3 below.

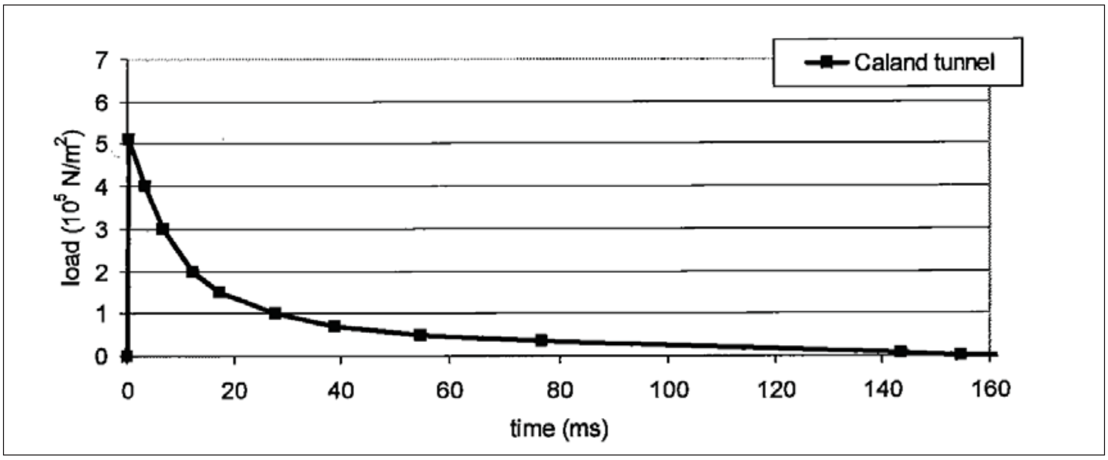


Figure 4-3 BLEVE load for the Caland tunnel according to TNO, copied from [20]

Apart from the explosion load, the deadweight of the cross-section is taken into account. Furthermore, permanent loads representing soil and water are included. These are considered to be mass less and are therefore apart from an initial displacement not of influence on the response to the dynamic load.

Schematisation of the structure:

- The structure was modelled numerically by means of the finite element code DIANA.
- The inner spans of the cross-sections instead of the system lines were used in order to prevent being too conservative.
- A mesh of 0.6 x 0.6 m was defined in which the cross-section was evaluated.
- The concrete as well as the applied reinforcement was modelled.
- Both linear and non linear material behaviour were embedded in the model.
- Interaction between the cross-section and the surrounding soil, like additional stiffness or damping was neglected.

Analysis

Calculations assuming linear elastic material behaviour as well as assuming non linear material behaviour were performed. There are significant differences between the results of these analyses, though it can be concluded that for a first indication of the response the assumption of linear elastic material behaviour is suitable.

4.2.2 Single degree of freedom 1 mass spring system

In order to investigate the interaction between the tunnel structure and the dynamic load, it is attempted to make a simplified analytical analysis, avoiding complex and time consuming modelling. If an explosion occurs inside one of the tunnel tubes, the structure is suddenly exposed to a large pressure. As a result, displacements will occur, whereas the structure deforms. The resistance against this deformation is dependent on the flexural stiffness of the different elements which are the roof and floor slab and the outer and intermediate walls. The behaviour of the elements can be investigated by schematizing these as mass spring systems that are loaded by a force that varies in time, representing the explosion. In figure 4-4 this principle is presented schematically.

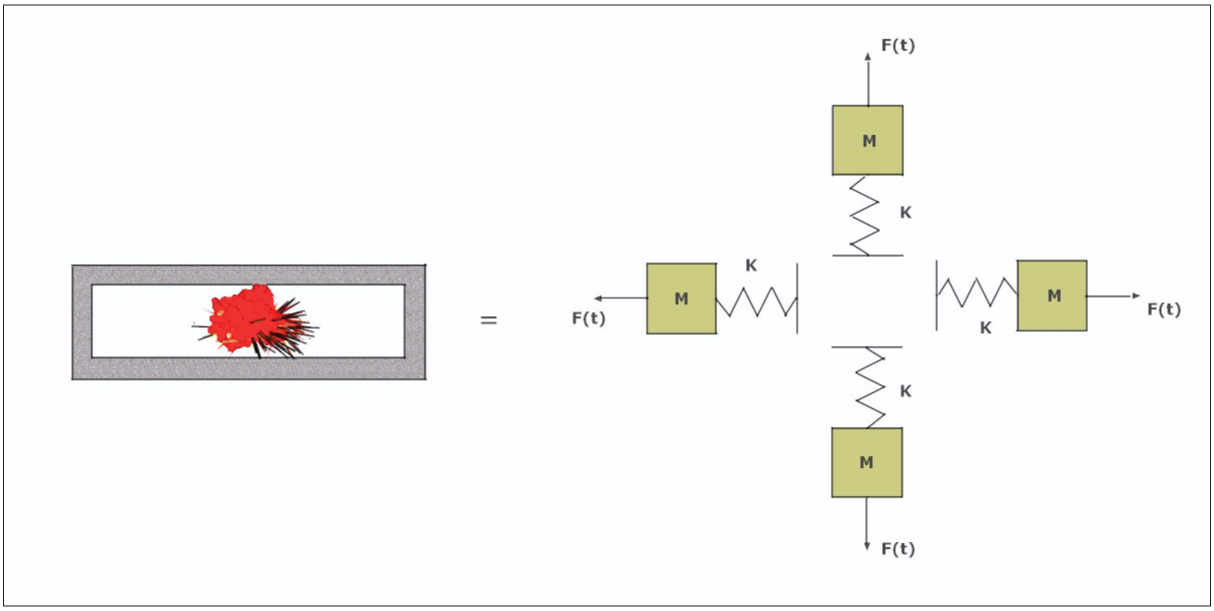


Figure 4-4 Schematisation by means of mass spring model

Schematization of the load

In case of an explosion, a relatively high pressure develops and also disappears in a very short time frame. Two extreme representations for this type of load are graphically presented in the figure below.

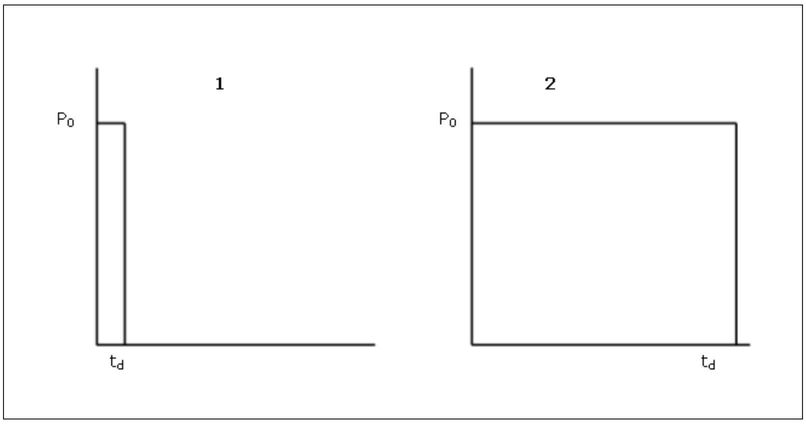


Figure 4-5 Extreme load cases

In the first case, a pulse load with a high magnitude, P_0 is suddenly present, the magnitude remains constant till it becomes 0 instantly after the duration of the explosion, t_d.
The second case concerns a load that also develops instantly though the duration is relatively long.

In reality the load is somewhere in between these extreme cases. If the duration of the loading time should be classified as 'long' or 'short' is dependent on the structural element that is considered. Each element has a natural period which is of influence to the response. This will be explained in more detail later. For now it is sufficient to realise that the ratio between this natural period and the duration of the load determines the way an element responds to the load.

For further analysis, the following schematisation will be applied, which is in between the extremes.

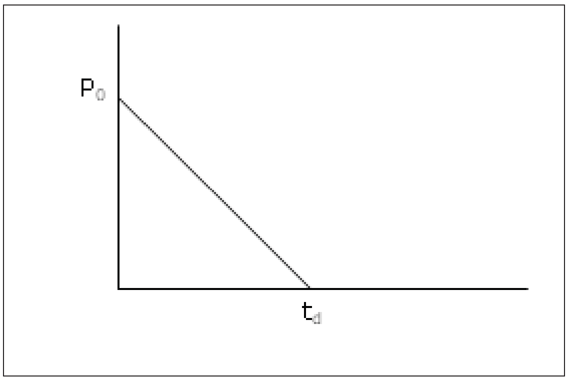


Figure 4-6 Schematisation of the explosion load

P_0 corresponds to the pressure that is suddenly present, whereby it is assumed that this pressure develops infinitely fast. It is presumed that the load will decay linearly during the load time, which is the duration of the explosion, t_d. Mathematically the load can be described as follows.

$$F(t) = P_0 \cdot \left(1 - \frac{t}{t_d}\right) \tag{4.1}$$

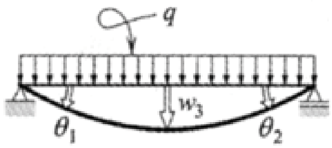
It is assumed that the pressure is equally distributed for all elements, whereas the magnitude of the load varies in time as stated in (4.1).

Schematization of the structure

As stated before, the structure will be divided into four elements, for the analysis these elements will be considered as beams with a width of 1 meter. The roof slab and intermediate wall are assumed to be the most vulnerable elements, for completeness, the outer walls and floor slab will be also considered.

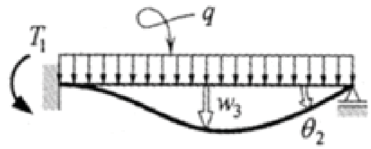
Stiffness

The spring constant for each element can be determined by means of relations from structural mechanics that describe the deflections as a function of the properties of element, type of supports and load. The load will be schematized to be equally distributed. For the analysis three different relations will be used, these are listed below.



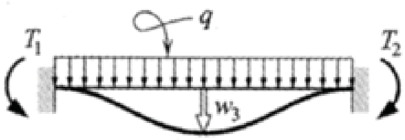
$$w_{3;\text{simply supported}} = \frac{5}{384} \cdot \frac{ql^4}{EI} \tag{4.2}$$

$$k = \frac{384}{5} \cdot \frac{EI}{l^4} \tag{4.3}$$



$$w_{3;\text{clamped-simple supported}} = \frac{1}{192} \cdot \frac{ql^4}{EI} \tag{4.4}$$

$$k = \frac{192 \cdot EI}{l^4} \tag{4.5}$$



$$w_{3;\text{clamped-clamped}} = \frac{1}{384} \cdot \frac{ql^4}{EI} \tag{4.6}$$

$$k = 384 \cdot \frac{EI}{l^4} \tag{4.7}$$

Where:

w	= The displacement	[m]
q	= The load	[kN/m]
k	= The spring constant	[N/m^2]
E	= The modulus of elasticity of the material	[N/m^2]
I	= The moment of inertia	[m^4]
l	= The span of the element	[m]

Damping
Since the load due to an explosion has a very short duration, it is expected that the maximum displacement will occur during the first vibration, whereby damping is of minor importance, therefore it is not taken into account for this analysis.

Mass
For the mass, the deadweight of the elements will be used. Since usually a surcharge load and surrounding soil are present, it should be justified to take into account additional mass for the roof and floor slab and outer walls, for now this is neglected however, which is a safe approach at least.

$$m = \rho_{concrete} \cdot t \cdot l \cdot b \tag{4.8}$$

Where

m	= Mass	[kg]
ρ_{concrete}	= The density of concrete	[kg/m^3]
t	= The thickness of the element	[m]
l	= The length of the element	[m]
b	= The considered width =1	[m]

Equation of motion

The equation of motion for a single degree of freedom mass spring system can be described as follows.

$$m\ddot{u} + ku = F(t) \tag{4.9}$$

The corresponding mass spring system for an element exposed to the triangular shaped explosion load can be described by the following relation.

$$m\ddot{u} + ku = P_0 \cdot \left(1 - \frac{t}{t_d}\right) \tag{4.10}$$

Analysis
In order to find a solution for the response of the defined system, the convolution integral can be used. Basically, with this method the load can be described as a collection of unity pulses with an infinitesimally small duration.

Theoretical background unity pulse

To start with, the nature of the unity pulse will be explained. Consider a very large force that acts on a structure in a very short time frame. A suitable mathematical tool for this type of problems is the delta function. This function is defined to have an infinite value at t=0, whereas it is 0 for any other value of t. Besides, by definition the following holds.

$$\int_{0^-}^{0^+} \delta(t)dt = 1 \tag{4.11}$$

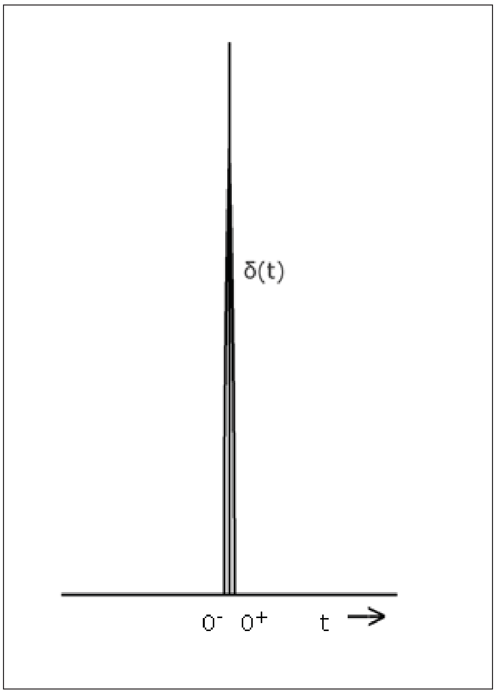


Figure 4-7 Pulse load

The load can be defined as follows.

$$F_u(t) = S_0\delta(t) \tag{4.12}$$

Where

S_0 = Pulse [Ns]

With Newton's second law, the following result can be obtained.

$$\int_{0^-}^{0^+} F_u(t)dt - \int_{0^-}^{0^+} F(t)dt = mv(0^+) - mv(0^-) \tag{4.13}$$

The amount of impulse that is transmitted during the short time frame can be described as follows.

$$\int_{0^-}^{0^+} F_u(t)dt = \int_{0^-}^{0^+} S_0\delta(t) = S_0 \tag{4.14}$$

This impulse results in a finite velocity, integration for the domain (0^-, 0^+) results in a negligible translation and therefore negligible spring force at t = 0^+. With (4.13) the following result can be obtained.

$$mv(0^+) - mv(0^-) = S_0 \tag{4.15}$$

At t = 0^- the system is at rest, so for the velocity at t = 0^+ is equal to

$$v(0^+) = S_0/m \tag{4.16}$$

Given the fact that there is no external force at t=0^+, the following differential equation can be used for a mass spring system.

$$m\ddot{u} + ku = 0 \tag{4.17}$$

The solution to this homogeneous differential equation is indicated below.

$$u(t) = C\cos(\omega_0 t + \phi) \tag{4.18}$$

Where

$$\omega_0 = \sqrt{k/m} \tag{4.19}$$

In which

C	=	Constant	[-]
ω_0	=	The natural angular frequency	[rad/s]
ϕ	=	The phase constant	[-]
k	=	The springconstant	[N/m]
m	=	The mass of the element	[kg]

The following initial conditions apply.

$$u(0^+) = 0 \tag{4.20}$$

$$v(0^+) = S_0/m \tag{4.21}$$

With these conditions, the following solution is obtained.

$$u(t) = \frac{S_0}{m\omega_0}\sin(\omega_0 t) \tag{4.22}$$

For the unity pulse, the case that $S_0 = 1$, the response is given by

$$h(t) = \frac{1}{m\omega_0}\sin(\omega_0 t) \tag{4.23}$$

Convolution integral

The foregoing theory can be applied to describe the triangular shaped load and can be described with an infinite collection of unity pulses, h(t). This principle is presented graphically in figure 4-8.

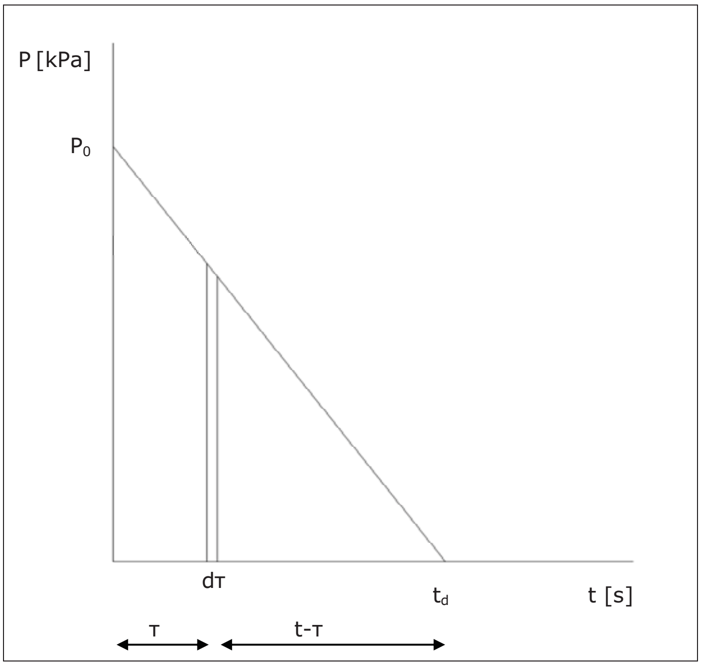


Figure 4-8 Schematizing the load by a collection of pulses

The load as a result of the unity pulse can be described as follows.

$$dS_0 = F(\tau)d\tau \tag{4.24}$$

The response of the system at t, as a result of a unity pulse that is present at t= τ can be described as follows.

$$du = dS_0 h(t-\tau) \tag{4.25}$$

Between t=0 and t an infinite amount of unity pulses is present. The contribution of each unity pulse should be summed in order to find the response of the system. Mathematically this can be described with the following integral.

$$u(t) = \int_0^t F(\tau)h(t-\tau)d\tau \tag{4.26}$$

Since the duration of the explosion is limited by t_d, the integral transforms into

$$u(t) = \int_0^{t_d} F(\tau)h(t-\tau)d\tau \tag{4.27}$$

The initial conditions can be applied.

Initially there is no displacement	$u(0) = 0$
Initially there is no movement	$\upsilon(0) = 0$

Elaboration results in the following expression for the displacement.

$$u(t) = \frac{P_0}{k}\left(1 - \frac{t}{t_d} - \cos\omega_n t + \frac{1}{\omega_n t_d}\sin\omega_n t\right) \qquad \text{for } t \le t_d \qquad (4.28)$$

$$u(t) = \frac{P_0}{k}\left(\frac{\sin\omega_n t}{\omega_n t_d} - \frac{\sin\omega_n (t - t_d)}{\omega_n t_d} - \cos(\omega_n t)\right) \qquad \text{for } t \ge t_d \qquad (4.29)$$

In this equation, ω corresponds to the natural frequency of the element. Since the explosion is short, it is assumed that the response will be in the first natural mode. For beams with a constant flexural stiffness, constant cross-section and density, the natural frequencies can be described with the following relation. The mass, represented by the deadweight of the element is accounted for in this expression.

$$\omega_n = C\sqrt{EI/\rho Al^4} \qquad (4.30)$$

Where:

ω_n	= The natural frequency	[rad/s]
C	= Constant	[–]
E	= The modulus of elasticity	[N/m^2]
I	= The modulus of inertia	[m^4]
ρ	= The density of the material	[kg/m^3]
A	= The cross sectional area	[m^2]
l	= The length of the element	[m]

For the first natural mode, n=1, the values for the relevant schematisations are presented in figure 4-9

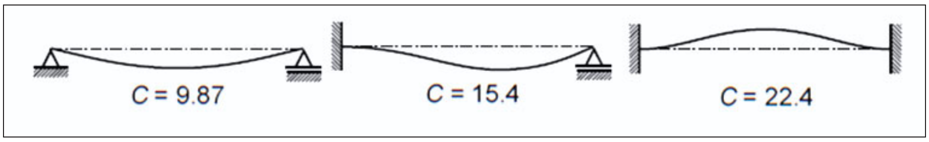


Figure 4-9 Values of C for different schematisations

The software package MAPLE is used to perform calculations. In order to validate the model, it will be compared to the results of the numerical analysis to the effect of explosions, done by TNO in 2007 as discussed in paragraph 4.2.1.

4.2.3 Comparison of the model by TNO and the mass spring model

In order to investigate the response of existing tunnels to explosion loads, TNO developed a numerical model using the software package DIANA. The structural behaviour of the Caland tunnel and Drecht tunnel in case of a BLEVE was considered. The results of the research can be compared with the analysis by means of a mass spring system. The same dimensions and material data are used. It should be noted that the model by TNO is rather sophisticated and many parameters are taken into account whereas the problem is simplified to a large extent in the mass spring model.

A number of important characteristics of both methods are listed in table 4-1.

Model TNO	Mass spring model
Numerical model	Analytical model
Bending is considered to be normative	Bending is considered to be normative
Both linear elastic and non linear behaviour	Linear elastic behaviour
Exponentially decaying load	Triangularly shaped load
No damping taken into account	No damping taken into account
Surcharge load taken into account	No surcharge load taken into account
Un cracked modulus of elasticity used	Un cracked modulus of elasticity used
Floor slab not considered	Floor slab considered

Table 4-1 Comparison model TNO - Mass spring model

- The analysis by TNO was performed by means of the finite element software package DIANA. A mesh is defined and the response of the structure is determined numerically. The mass spring model is of analytical nature, using the laws from structural dynamics and structural mechanics.

- For both models it is assumed that bending is normative for the capacity. Failure as a result of exceeding of the shear capacity is not considered.

- The evaluation by TNO was done assuming linear elastic material behaviour as well as non linear behaviour. For the validation of the mass spring model only the results of the linear elastic analysis will be compared. The results of the non linear calculations are different but in the same order of magnitude, which supports the relevance of a simpler, linear elastic calculation for comparison of alternatives and acquiring an indication of the response of the structure to an explosion load.

- Since the explosion load is partly dependent on the geometry of the cross-section, for each considered tunnel a normative load case was defined by TNO. For evaluation with the mass spring model these loads are schematised to be of triangular shape, in such a way that the peak pressure and impulse are equal. The impulse corresponds to the area under the curve. The load as it will be used in the mass spring model in this paragraph is indicated figure 4-10 below.

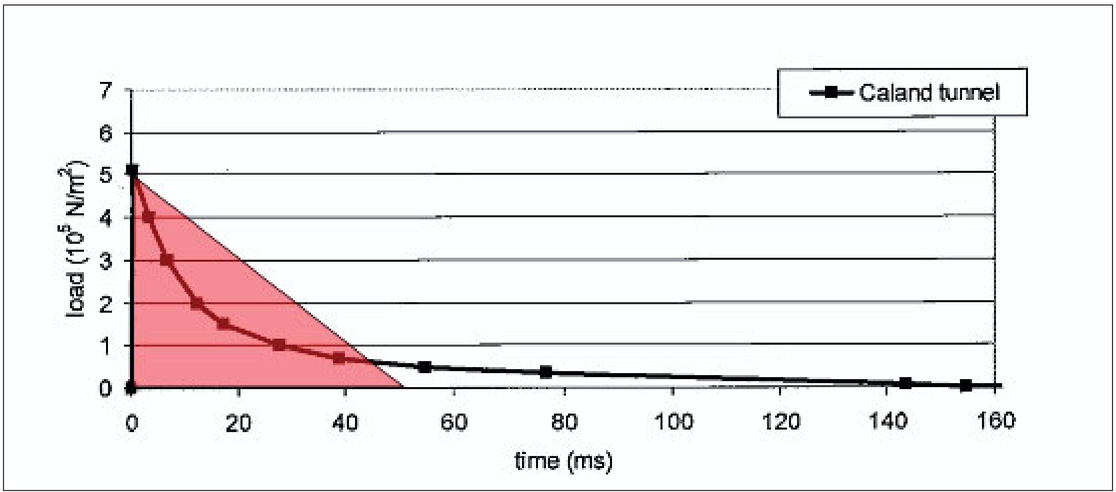


Figure 4-10 Schematized load (modified from [20])

Where P_0= 500 kPa and t_d = 0.05 s.

- An exponentially decaying load gives a better reflection of reality, though the assumed schematisation should be at least on the safe side. A higher pressure with equal impulse gives a less favourable situation for the structure, as can be also concluded from [11].

- The precise magnitude of the peak pressure and duration are not of interest for now, these will be considered later on. The purpose of this evaluation is to check the applicability of the mass spring model.

- Material damping as well as damping by the surrounding subsoil is neglected in both models. This assumption is at the safe side. It implies also that the model describes the behaviour of the intermediate walls more accurately than the rest of the structure, since no surrounding soil and water is present for these elements.

- In the analysis by TNO a surcharge load is taken into account whereas this is neglected in the mass spring analysis. The surcharge load that TNO accounted for, is however considered to be mass less. Therefore the presence of the surcharge load should not be of influence on the response. There may be an initial displacement due to this surcharge load in the roof however. The behaviour of the intermediate walls as described by the mass spring system will probably correspond better to the results of TNO than for the rest of the structure.

- In both models the uncracked modulus of elasticity is used. This seems plausible since the duration of the load is very short.

- In the analysis by TNO it was assumed that the floor slab is not vulnerable for explosion loads. Therefore, this element was not considered. The floor is elastically supported and has usually a relative large thickness, in the final situation a layer of ballast concrete is present, which increases the capacity also. Therefore, the assumption seems plausible. For completeness, the floor slab will be considered in the mass spring model however.

Model for the Caland tunnel

As stated before, the dimensions and material characteristics as used by TNO are also used for the mass spring model in order to make a good comparison. The parameters can be found in table 4-2 and figure 4-11.

Material	B35	
ρ_{B35}	2500	[kg/m³]
$E_{B35,\ un\ cracked}$	3.10E+10	[N/m²]

Table 4-2 Material data

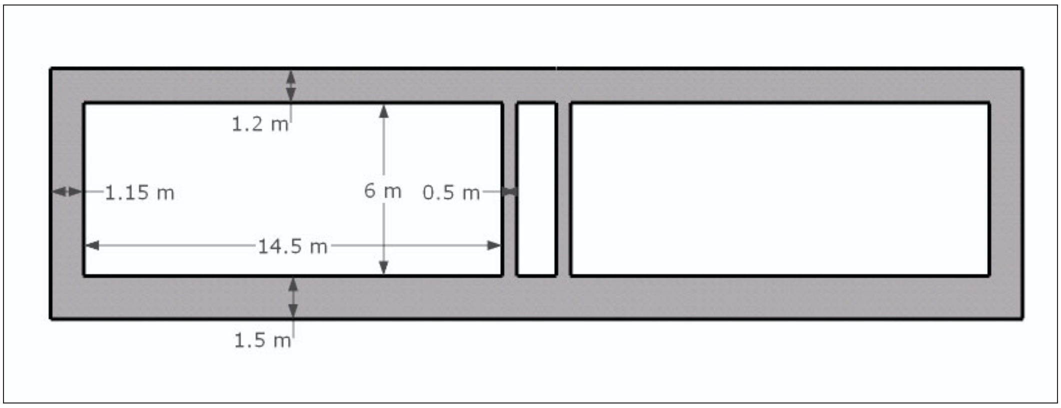


Figure 4-11 Main dimensions cross-section Caland tunnel

It is presumed that the schematisation as indicated in figure 4-12 corresponds best to reality. Since the schematisation may be of large influence on results of this analysis, it is decided to consider also different schemes for each element.

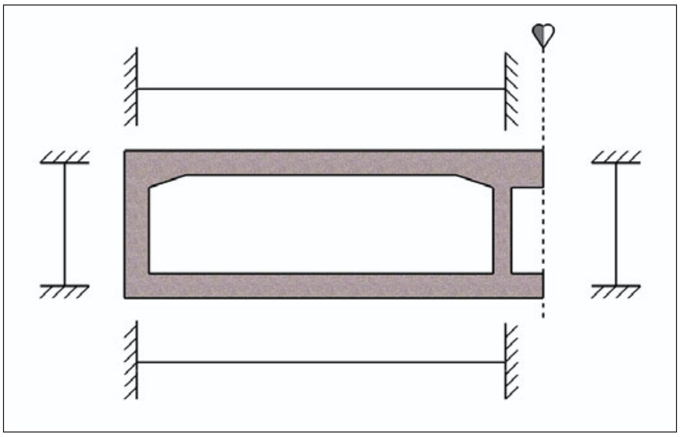


Figure 4-12 Schematisation of the structure

- The roof slab is presumed to be clamped supported at both sides. This seems plausible, since the slab continuous beyond the emergency corridor. Furthermore, the thickness near the supports is increased, making the connection stiffer. In order to investigate the influence of the schematisation also a clamped-simply supported and completely simply supported roof slab will be evaluated.

- Since the floor slab is supported linear elastically by the subsoil, it will behave rather stiff. For completeness, the floor slab will be considered with all three schematizations, for which the same reasoning as for the roof slab holds. It is however expected that even the clamped –clamped schematisation is conservative.

- The intermediate wall is schematised to be clamped at both sides, which seems plausible due to the fact that the thickness of his element is usually small compared to the thickness of the roof and floor slab. For completeness the element will be also considered to be simply supported.

- The outer wall will be schematized to be clamped at both sides. For completeness also a simply supported scheme will be considered.

The calculations for the mass spring model are made with the software package MAPLE. For each element different static schemes are considered as stated before. The results are listed in table 4-3 whereas values for the earlier presumed representative schemes, indicated in figure 4-12 are printed bold. The displacements in outward direction, u_{max}, the maximum displacements inward, u_{min} , as well as the frequency of the response are determined.

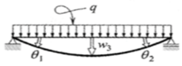
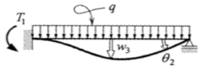
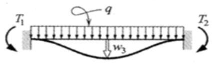
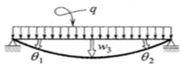
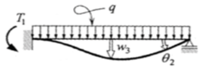
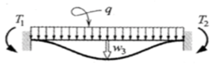
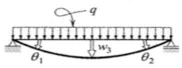
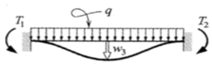
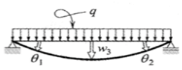
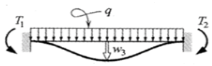
Element	Schematization M-S model	Mass spring model			TNO		
		U_{max} [mm]	U_{min} [mm]	f [hz]	U_{max} [mm]	U_{min} [mm]	f [hz]
Roof slab		72	-72	8	17*	-8*	18
Roof slab		36	-36	15	17*	-8*	18
Roof slab		**20**	**-12**	**21**	**17***	**-8***	**18**
Floor slab		44	-44	9	-	-	**-**
Floor slab		20	-15	15	-	-	-
Floor slab		11	-5	25	-	-	-
Intermediate wall		30	-20	20	6	-6	50
Intermediate wall		**7**	**-4**	**30**	**6**	**-6**	**50**
Outer wall		4	-2	25	2	-3	80
Outer wall		**<1**	**>-1**	**60**	**2**	**-3**	**80**

Table 4-3 Comparison between calculated response with mass spring model and the results of TNO

* An initial displacement is present for the roof, as indicated in figure 4-13, this is taken into account in the listed displacements.

- From the results, listed in the table above, it can be concluded that the schematisation of the structural elements is of great importance to the deflections and frequency of the response. Schematisation should therefore be made with care.

- The displacements for the assumed schematisation of the roof slab correspond very well to the results of TNO. Therefore, it is decided that the assumed scheme can be considered as representative for the roof slab. It should be noted that in the results of TNO an initial displacement is present. The reason for this is the presence of a surcharge load. In the mass spring model no initial displacements are present. In order to compare the effects of the explosion, the deflections relative to the initial situation are listed.

- Since the floor slab was not considered in the analysis by TNO, no comparison can be made for this element. It is assumed that the structural behaviour is comparable to the roof slab and therefore the same schematisation will be considered to be representative.

- The scheme that is assumed for the intermediate wall corresponds best to the results of TNO. The schematisation to simply supported beam will lead to displacements that are significantly too large.

- The results for the assumed scheme of the outer walls comply very well with the results of TNO. Considering these elements as clamped beams would hardly result in any displacement.

- Based on this evaluation, there is no need for adaptations of the presumed representative schematisation of the cross-section, as indicated in figure 4-12.

- The deviations between the two models are listed in table 4-4, the displacements are given in mm.

Model	u_{max} roof	u_{min} roof	u_{max} outer wall	u_{min} Outer wall	u_{max} intermediate wall	u_{min} intermediate wall
TNO	17	-14	2	-3	6	-6
M-s	20	-12	<1	>-1	6	-4
Deviation	17%	16%	100%	300%	0%	50%

Table 4-4 Deviations between the calculated deflections

Although deviations occur, especially for the outer wall, this is a pretty good result regarding the rather simple schematisation. Furthermore, the deflections are in the order of centimetres only whereas the deviations are in the order of millimetres.

- The results of the analysis performed by TNO are presented graphically in the figure below in which wall 1 corresponds to the outer wall and wall 2 corresponds to the intermediate wall.

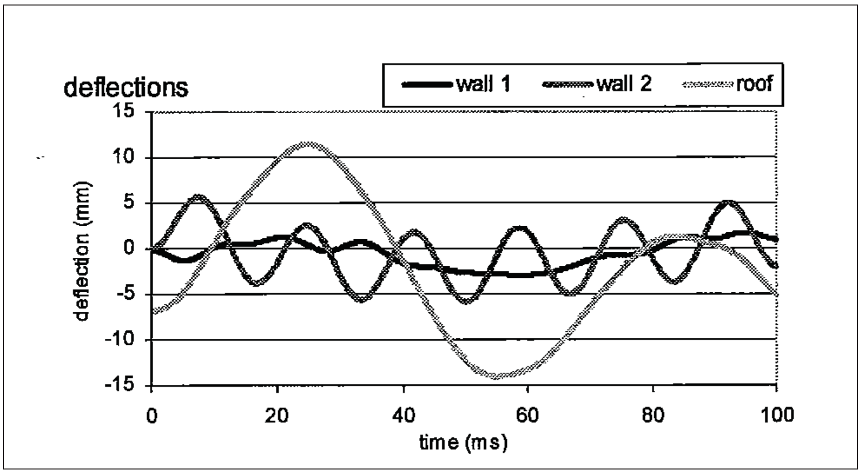


Figure 4-13 Results linearly elastic calculation TNO for the Caland tunnel [20]

- The results for the mass spring calculation for the representative considered schematisation of the elements is presented in figure 4-14.

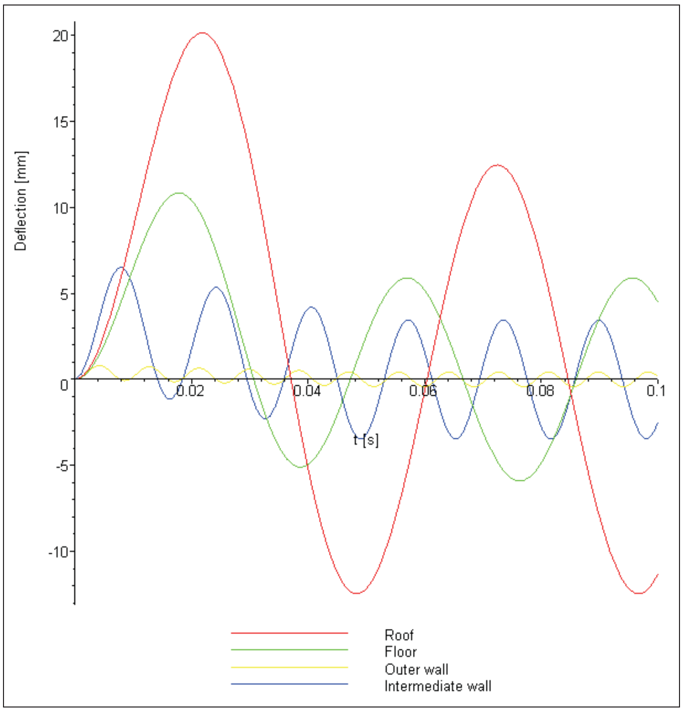


Figure 4-14 Results linearly elastic calculation mass-spring model

- The frequencies of the response of the two models comply very well. Since the deflection is dependent on the natural frequency, this is expectable. There are slight differences between the found frequencies, which is probably the result of the schematisation of the cross-section into separate elements for the mass spring model and a different schematisation of the load.

Model	f_{roof} [hz]	$f_{outer\ wall}$ [hz]	$f_{Intermediate\ wall}$ [hz]
TNO	18	80	50
Mass Spring model	21	60	30

Table 4-5 Deviation in frequencies

- It should be noted however that neither the mass spring system nor the model by TNO gives an accurate reflection of reality. Since except for the deadweight no mass is taken into account in both models, the natural frequencies that are obtained will deviate from reality. The surcharge load is usually of a large magnitude compared to the deadweight. It can however be stated that the results are at the safe side.

- Interaction between the structural elements is not taken into account in the modelling.

4.2.4 Sensitivity analysis mass spring model

The individual influence of the parameters of the mass spring method should be investigated in order to evaluate the sensitivity of the model. To this end, the case study Caland tunnel will be considered again. Basically, there are two aspects that determine the response, these are the load to which the element is exposed and the dimensions of the element. These will be varied, whereby the static scheme will be as assumed earlier, which is supported by the foregoing calculations. For this analysis only the deflections will be considered since these are for design purposes of major interest.

Load

The load is schematised to be of triangular shape, and is defined by the parameters P_o and t_d. Both parameters are varied within a plausible range, keeping all remaining data unchanged in order to investigate the individual influence. For t_d a range between 0.01 and 1 s will be considered. The Peak pressure will be varied between 100 kPa and 1000 kPa. Since the deflections that are found with the mass spring model are approximately of equal magnitude in both directions, the absolute value will be presented in the following figures.

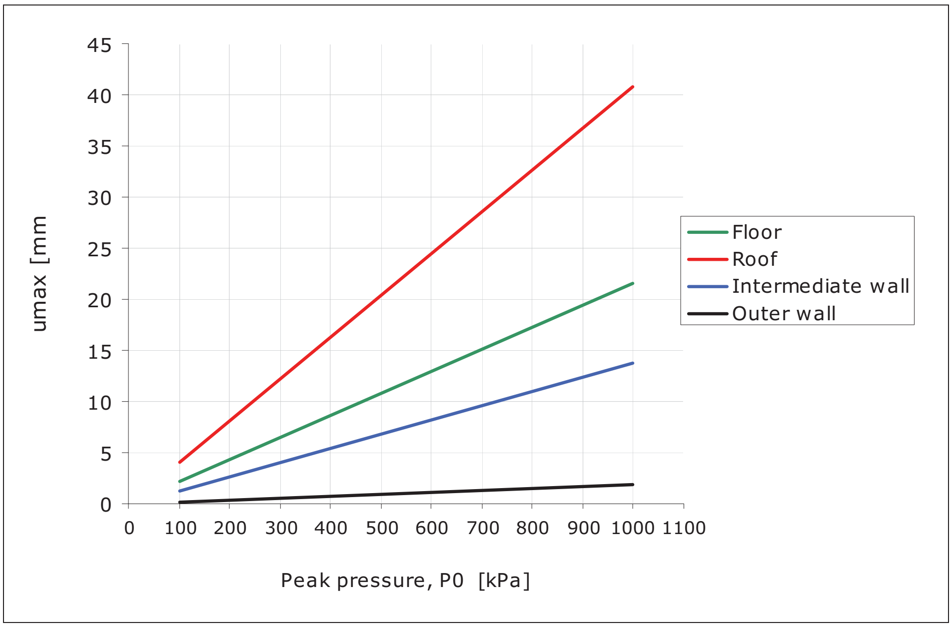


Figure 4-15 Influence of the peak pressure to the maximal deflection for the mass spring model

There exists a linear relation between the occurring peak pressure and the deflection of the elements. Which is trivial, considering equations (4.28) and (4.29).

The influence of the duration of the load, t_d, to the response is presented in figure 4-16.

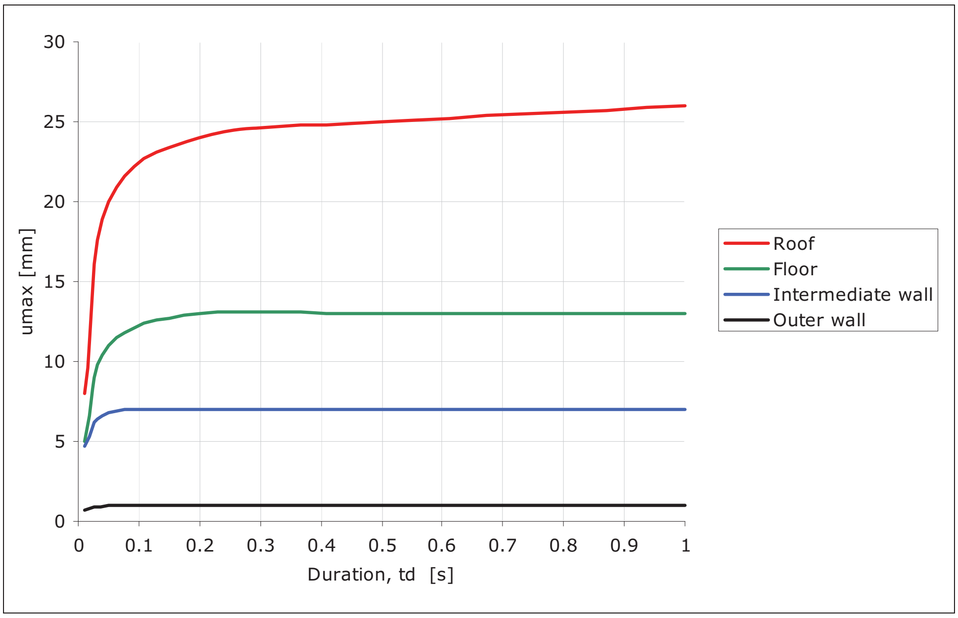


Figure 4-16 Influence of the duration of the load on the maximal deflection mass spring model

The maximal displacement is dependent on the duration of the load, from a certain moment the displacement will not further increase as can be concluded from the figure above. This can be explained by the fact that for duration of the load, td that is significant longer than the natural period of the element, a response similar as for a block load will occur. The structural element experiences an almost constant load instead of a linear decaying as indicated in figure 4-17.

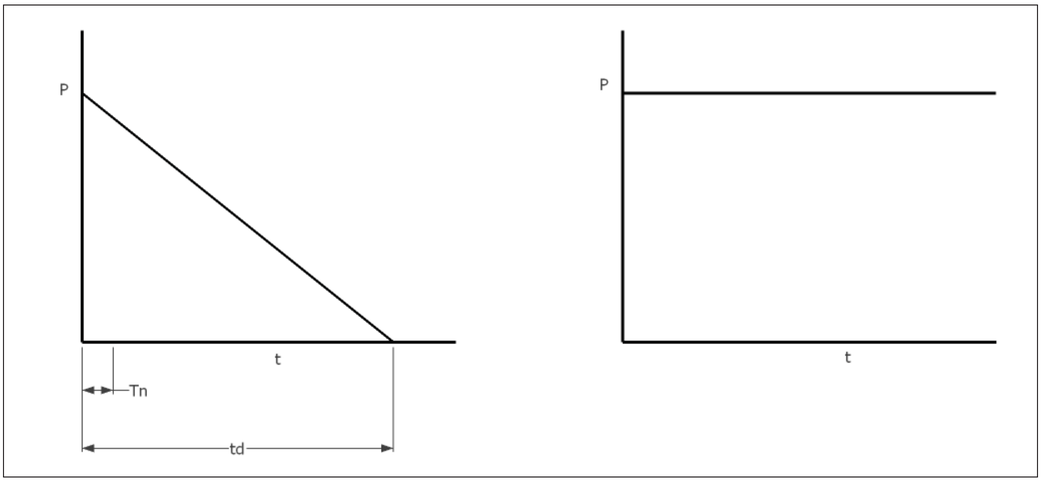


Figure 4-17 Schematisation of the load for td >> Tn

Since the structural element experiences a load that has a long duration, t_d the equation of motion reduces to.

$$u(t) = \frac{P}{K}(1 - \cos \omega t) \tag{4.31}$$

Obviously, the maximal displacement is equal to 2 times the statically displacement in that case. This corresponds to the results presented in figure 4-16.

Dimensions
The thickness of the elements will be varied within the ranges that are indicated in the table below.

Element	Range thickness [m]	Range length [m]
Roof	0.8 - 2	12 - 20
Floor	1 – 2.5	12 - 20
Intermediate wall	0.5 – 1.5	5 - 10
Outer wall	0.8 – 1.5	5- 10

Table 4-6 Ranges for the investigated parameters

- For the thickness of the roof 0.8 m is considered to be the minimum. Since an explosion introduces severe forces the maximal value to be considered is chosen to be 2 m.

- The floor slab has usually a slightly larger thickness than the roof slab. Therefore, the range that will be considered for the thickness of this element is 1 – 2.5 m.

- An absolute minimal value for the thickness of the intermediate walls is considered to be 0.5 m. If explosion loads have to be taken into account, it is very likely that the thickness should be increased to a large extent, therefore the upper limit is chosen to be 1.5 m.

- Under normal circumstances the outer walls have a greater thickness compared to the intermediate walls, due to the presence of water and soil pressure. Therefore, the smallest considered thickness for these walls is chosen to be 0.8 m, whereas the upper limit is 1.5 m in correspondence with the intermediate walls.

In the figure below, the sensitivity to variations of the thickness is for each element, where all remaining data is unchanged is graphically presented in figure 4-18.

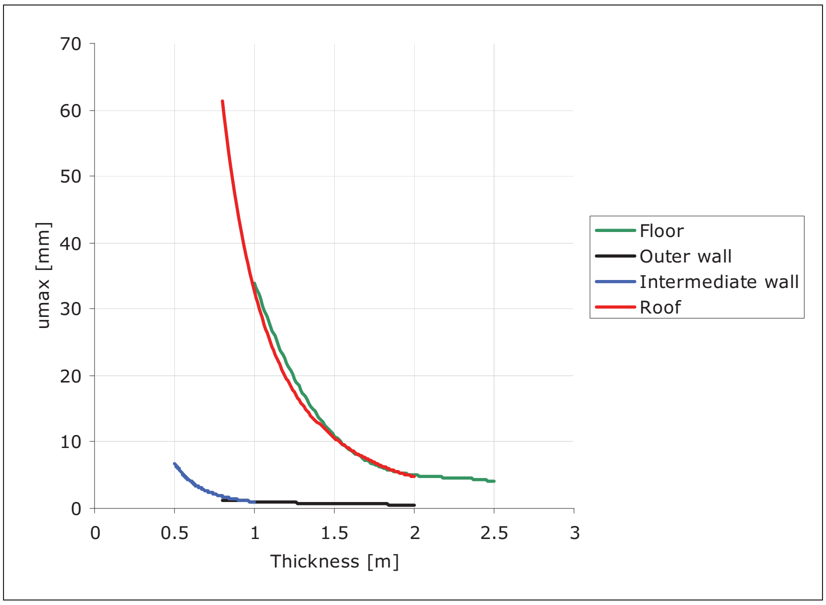


Figure 4-18 Influence of the thickness on the maximal deflection for the mass spring model

- It can be concluded that the thickness is of important influence to the maximal deflections. The effect is stronger for smaller thicknesses.

- It is clear that varying the thickness has an important influence on the displacements.

For the span of the elements, similar analysis is performed.

- The lower limit for the span of the roof is chosen to be 12 m, this is considered to be a minimum value. For the upper limit 20 m is used, since this is a practical maximum. Obviously, the same range holds for the floor slab.

- For the height of walls, 5 m is considered to be the minimum achievable value. Usually it is attempted to limit the height, therefore the maximum of 10 m is an unlikely choice, it is possible however.

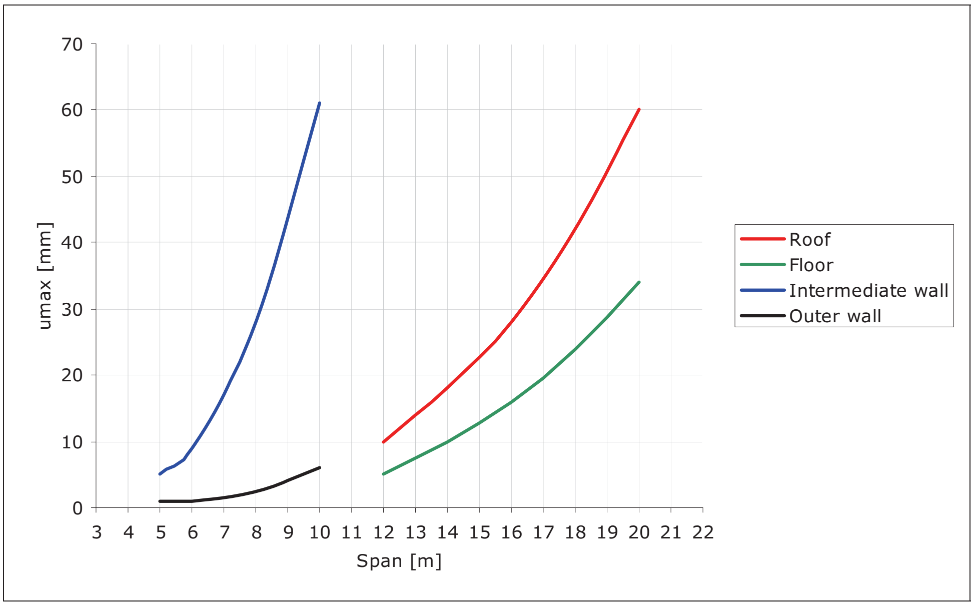


Figure 4-19 Influence of the span on the maximal deflection

There exists a strong relation between the span of the elements and the magnitude of the deflection. The slope of the lines is determined by the stiffness the schematisation of the element.

Additional mass

In the analysis so far, the only mass that is taken into account is due to the deadweight of the structure. In practice, there is usually a considerable surcharge load, consisting of water and soil present on top of the roof. For the outer wall also additional mass can be taken into account as a result of the horizontal soil and water pressure. The floor slab is exposed to upwards direct water pressure, which could be considered as additional mass. The intermediate walls are, however, not exposed to any permanent load that justifies the application of additional mass.

The effect of the surcharge load on the response of the roof slab is investigated with the mass spring model. For simplicity, only the additional mass due to the water is considered. This additional mass is taken into account by increasing the unit weight of the slab in such a way that it corresponds to various water depths.

The influence of the water depth to the response of the roof slab is depicted in figure 4-20. For the outer walls and floor slab similar results are obtained.

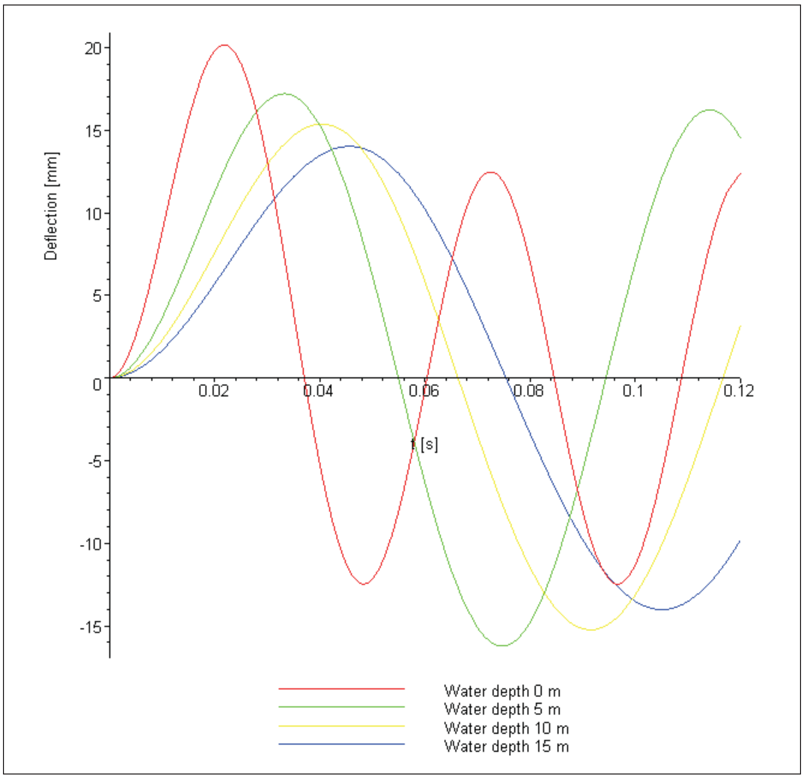


Figure 4-20 Influence of the water dept to the response of the roof slab

It can be concluded that the deflections decrease, whereas the frequency of the response also decreases. The presence of water on top of the roof may provide a significant reduction to the occurring outward directed displacements. The rebound or inward directed displacements are higher compared to the case no additional mass is taken into account, though for greater water depths this effect is of less magnitude.

4.2.5 Conclusions

- It can be stated that the schematisation for the considered case gives a pretty accurate order of the magnitude for the displacements as long as the concrete behaves linear elastically. Therefore, it can be used as a screening tool in early stages of design. Another application is preliminary investigation the effect of adaptations of the cross-section.

- It should be noted that in this rather simple method should be considered with care, since a number of aspects are not taken into account. First of all linear behaviour of the material is assumed, which is not necessarily the case under these extreme loading conditions. Besides, damping by the soil mass is neglected, as well as the permanent loads that act on the structure.

- In the analysis of the Caland tunnel by TNO, many aspects were taken into account. A simple analysis by means of a mass spring system results in comparable displacements.

- The results that are obtained with this approach are dependent on the geometry of the cross-section and the dimensions of the elements to a large extent.

4.3 *Quasi static approach*

For design purposes explosions are sometimes schematized by means of a static load. The benefit of this approach is that the calculations that have to be performed for the structural design are straight forward.

It is usual to define an outward directed static load as well as an inward directed load as indicated in figure 4-21.

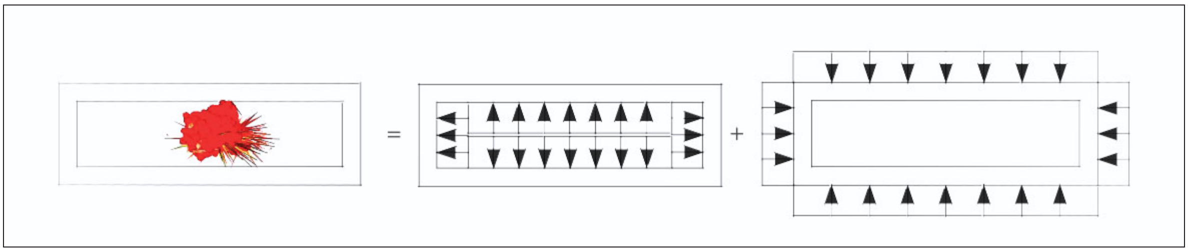


Figure 4-21 Quasi statically approach

To this end, it is interesting to determine the statically equivalent load, which is the static load that results in the same deflection as the dynamic load. The explosion load, that varies in time is thus translated into a static load as indicated in figure 4-22.

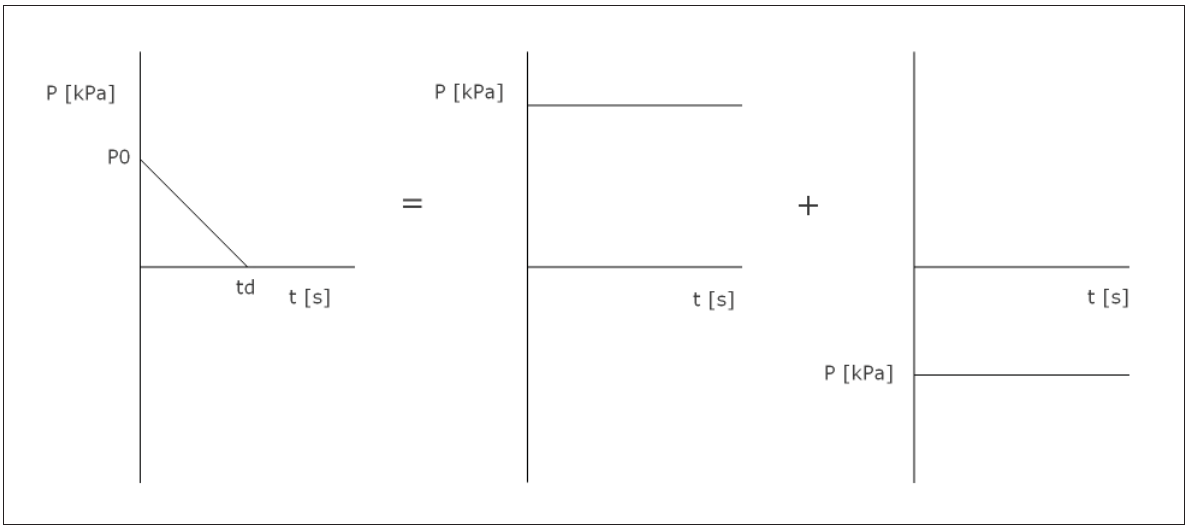


Figure 4-22 Translation of the dynamic load into two statically components

The ratio between the statically equivalent load and the dynamic peak pressure is the Dynamic Load Factor (DLF).

$$DLF = \frac{P_{statically\ equivalent}}{P_0} \tag{4.32}$$

The statically equivalent load can be calculated with the following expression.

$$P_{staticallyequivalent} = k \cdot u \tag{4.33}$$

The value of k can be obtained with equation (4.3), (4.5) or (4.7), depending on the schematisation and the displacement can be described with the following relation as derived earlier.

$$u(t) = \frac{P_0}{k}\left(1 - \frac{t}{t_d} - \cos\omega_n t + \frac{1}{\omega_n t_d}\sin\omega_n t\right) \qquad \text{for } t \le t_d \tag{4.34}$$

$$u(t) = \frac{P_0}{k}\left(\frac{\sin\omega_n t}{\omega_n t_d} - \frac{\sin\omega_n (t - t_d)}{\omega_n t_d} - \cos(\omega_n t)\right) \qquad \text{for } t \ge t_d \tag{4.35}$$

The DLF gives an indication of the response of an element to the dynamic load. The magnitude of this parameter varies between 0 and 2, since the maximum value of the part of expression for u within the brackets is 2. The term P_0/k in fact represents the displacement due to a static load with magnitude P_0.

The results that are obtained for the Caland Tunnel in the foregoing paragraphs will be used again for this evaluation

	P_0 [kPa]	Deflection according to mass spring model [mm]	$P_{Statically\ equivalent}$ [kPa]	DLF
Roof	500 ↑	20↑	776↑	1.6
		12↓	265↓	0.5
Floor	500↓	11↓	833↓	1.7
		5↑	379↑	0.8
Intermediate wall	500 →	7→	991 →	2
		4←	566←	1.1
Outer wall	500 ←	1←	1000←	2
		<1→	500→	1

Table 4-7 Statically equivalent loads and DLF's for the Caland tunnel

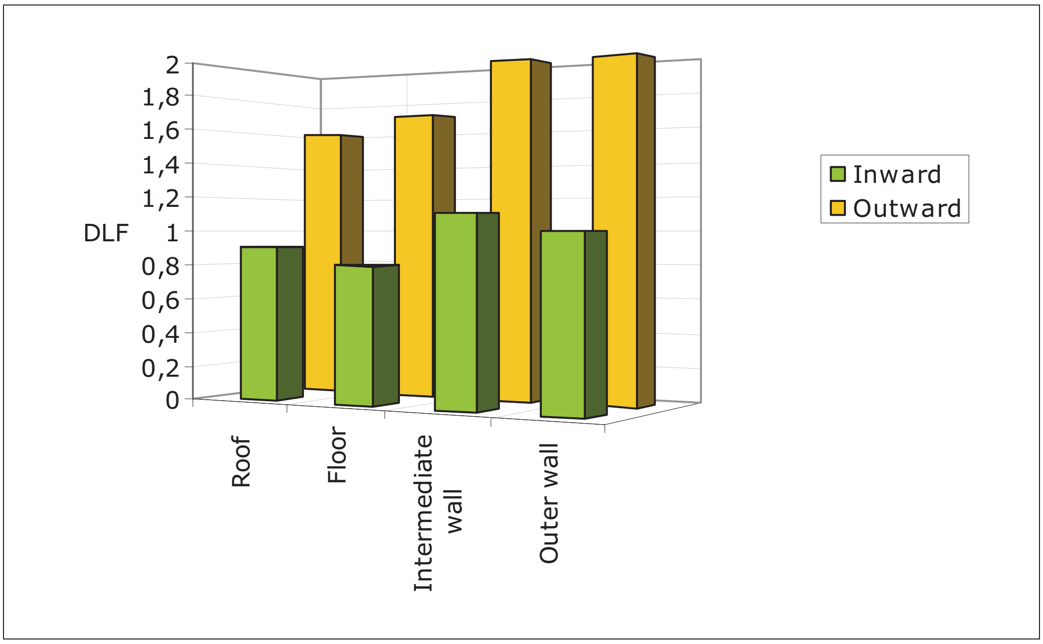


Figure 4-23 DLF's Caland tunnel

It should be noted that for the roof and floor a lower DLF is found for the outward directed load compared to the walls. The obtained results imply that the roof and floor are less vulnerable for the dynamic load. The differences are not very large, order of magnitude 30% and therefore the requirement for applying a constant static load seems plausible.
If the elements should be considered separately, the statically equivalent load that should be applied for the roof and floor slab can be lower than those for the walls. This implies that if a constant value should be applied, a difference in structural capacity between the several elements exists. For an efficient design, it would be desirable to have more or less equal capacity however. It makes no sense to design one particular element for an explosion that is much higher than the explosion that can be withstood by another element. Since the explosion load is unidirectional, the structure is as strong as the weakest element after all. This result is however found with a rather simplified approach and it should be noted that the response is also strongly dependent on the peak pressure and duration of the explosion. For practical reasons it is plausible that the requirement concerning explosion loads is stated as a constant value for the load.

The ratio between the DLF that should be applied for the inward directed load and the DLF for the outward directed load is listed for the different elements in table 4-8.

Element	DLF_{inward}/ $DLF_{outward}$
Roof	0.3
Floor	0.5
Intermediate wall	0.6
Outer wall	0.5

Table 4-8 Ratio between DLFs

The magnitude of the dynamic load factor is mainly determined by the ratio between the duration of the explosion load, t_d and the natural period of the structural element that is considered, as depicted in figure 4-24.

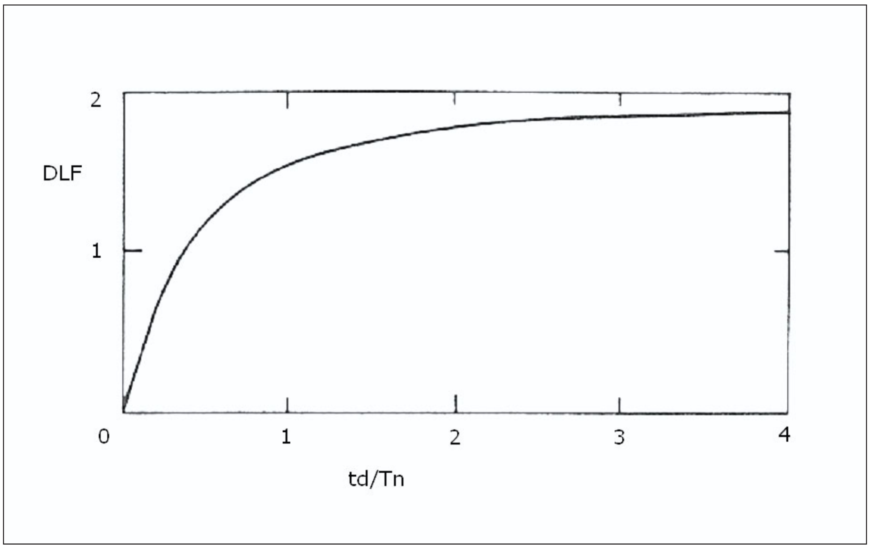


Figure 4-24 DLF as function of td/Tn

$$T_n = \frac{2\pi}{\omega_n} \qquad (4.36)$$

Where

T_n	=	The natural period of the element	[s]
ω_n	=	The natural frequency of the element	[rad/s]

The value for ω_n can be obtained with (4.30).
The surrounding soil and water is not taken into account in this model. The presence may however result in an increased natural period of the structural elements. The DLF's could be smaller in that case.

Conclusions

- It is important to realise that the occurring pressure is equal for all elements, but the elements respond in a different way to this load. Therefore, different statically equivalent loads are found. Since the explosion loads are of extreme magnitude, considering the roof and walls separately could be a beneficial approach.

- The dynamic load factor of an element is dependent on its dimensions. Therefore it may also be attractive to make adjustments in such a way that the element responds less intense to the load. The foregoing statements are however only applicable in practice if there is a framed domain with respect to the duration of the explosion and the occurring peak pressure.

- The response of the structure is strongly depended on the dimensions and geometry. If a static requirement is stated, it is not clear what the desired level of safety is. Different designs may all provide sufficient capacity to withstand a certain static load, though will respond quite different to the explosion. In fact, the transformation of the dynamic load into a static load is influenced by the dimensions of the exposed structure to a large extent. Therefore, just stating a static load appears to be a little too easy. Prescribing a peak pressure and impulse would be more plausible.

4.4 *Dynamic module of Plaxis*

The finite element code Plaxis is developed for soil and rock analysis. It is possible to model a structure with this program and perform analysis, taking into account interactions with soil and water. Since also dynamic calculations can be performed with the program, it may be a suitable tool in order to investigate the event of an explosion in an immersed tunnel.

4.4.1 Comparison with other models

In order to verify the applicability of the dynamic module, an exploring calculation is performed. The example of the Caland tunnel, discussed in paragraph 4.2.3 will be considered again. In order to make a good comparison with the results found by TNO, the same data and starting points are used. These are listed below.

- Linear elastic material behaviour is assumed
- The water and soil pressure is accounted for by means of massless distributed loads
- The influence of the soil is not taken into account
- The walls and roof slab are considered, whereas it is assumed that the floor slab is well supported and therefore less vulnerable to the explosion load.

The purpose of this calculation is to investigate if the dynamic response of the structure complies with the results found by TNO and the analytical mass spring model. The dynamic module of Plaxis, version 8.4 is used to model the case that is considered by TNO, a graphical presentation of the schematized structure is presented in figure 4-25.

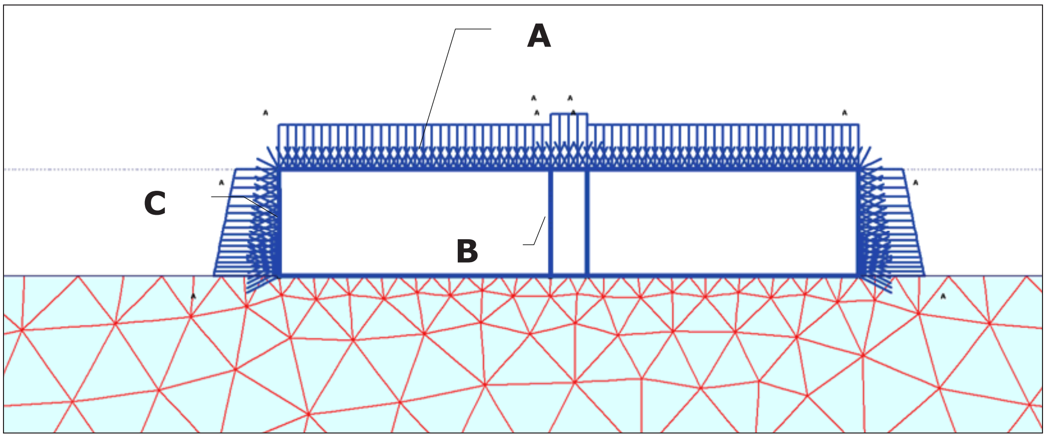


Figure 4-25 Plaxis model Caland tunnel

The explosion load can be taken into account by means of a unit load that is multiplied with a prescribed multiplier that is time dependent. The displacements for the points indicated in figure 4-25 are given in figure 4-26. For comparison, the results of TNO and the mass spring model for the same case are presented in figure 4-27.

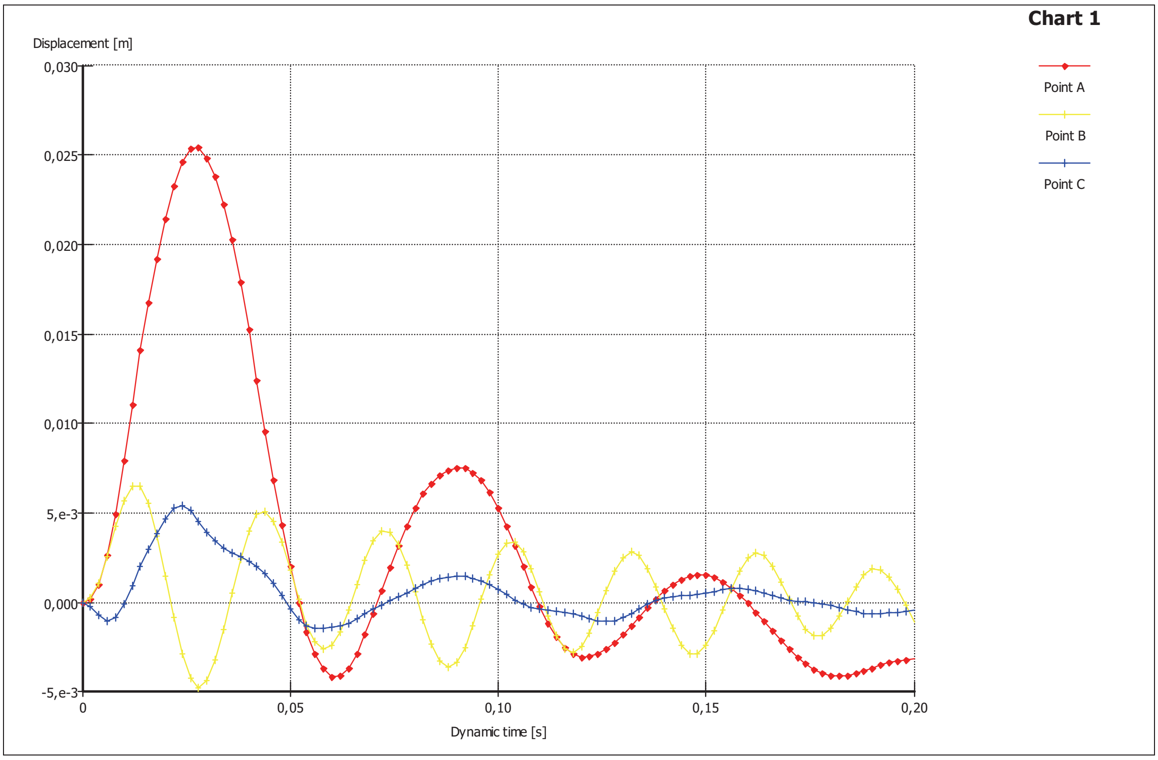


Figure 4-26 Results Caland tunnel Plaxis

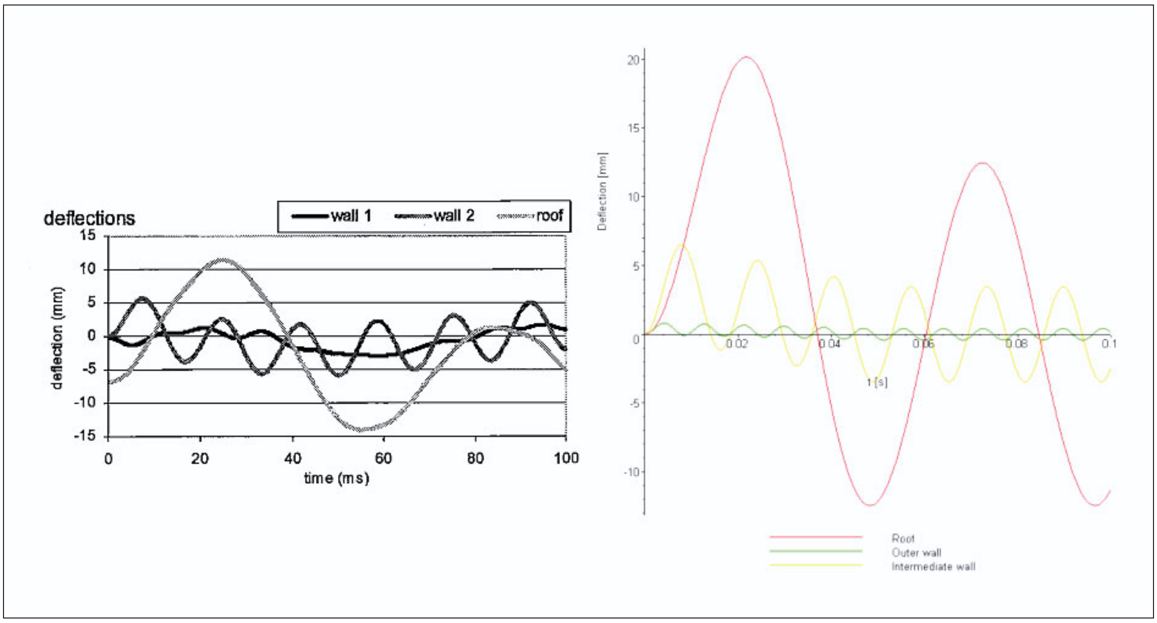


Figure 4-27 Results Caland tunnel TNO (left side) and mass spring model (right side)

It can be concluded that the response of the structure that is determined with Plaxis corresponds quite well to the results of the mass spring model and the results of TNO. The response of the roof slab according to the results of TNO differs slightly. The reason for this is an initial deflection due to the surcharge load, which is neglected in the mass spring system and Plaxis model. Since the surcharge load is modelled as a massless load by TNO, it has no influence on the response, apart from a initial displacement. In fact the curve representing the response of the roof is shifted downwards with a magnitude of the initial displacement. The response relative to this initial displacement corresponds very well with the other models.

As stated before, the considered case is simplified to a great extent. Although in this way it can be verified that the response of the concrete structure to the dynamic load is described in a suitable way with the dynamic module of Plaxis.

Plaxis is developed for soil analyses, therefore it is possible to take into account the influence of the surrounding soil and furthermore the phreatic level can be modelled. Extending the model with these aspects will result in a better representation of reality. Besides this, it is possible to model plastic material behaviour. To this end a plastic bending moment en axial force can be defined for each structural element.

4.5 *Review static requirement as stated for the Oosterweel tunnel*

The direct reason for this research is the fact that BAM Infraconsult participated in the tender phase for the Oosterweel tunnel, Antwerp, for which an extraordinary requirement concerning explosion loads was stated. One of the goals of this research is to verify this requirement.

The requirement that was stated concerned a static load of 500 kPa directed outwards and a static load of 300 kPa directed inwards. In this paragraph this load will be reviewed.

4.5.1 Representative load case

An explosion is a complex phenomenon, as stated before. The load on the structure as a result is dependent on various aspects, these are listed below.

- Peak pressure
- Duration of the explosion
- Cross-section

Within the framework of the research to the response of tunnels to this type of loads, TNO investigated the normative type of explosion and the accompanied magnitude. In an interview with Dr. Ir. J. Weerheijm, TNO Defence and Safety, in May 2008, the following is concluded about this research.

- The representative load is due to a LPG BLEVE of a vessel containing 50 m^3, T = 326 k.
- The rupture of the vessel is very short, and should be considered to occur instantaneously.
- For the analysis a circular cross-section of 72 m^2 is assumed.

It is concluded that two zones can be distinguished.

- The occurring dynamic peak pressure, P_0 according to the developed model is equal to 1300 kPa at the first 20 meter from the centre of the BLEVE.
- The Impulse, I is equal to 52 kPa.s for the first 20 meter from the centre of the BLEVE.
- Beyond 20 meter from the centre of the BLEVE a peak pressure of 150 kPA applies.
- The impulse beyond 20 meters from the centre of the BLEVE is 18 kPa·s.

The analysis is made for a circular cross-section, it is considered to be representative for a rectangular cross-section as well. In consultation with Dr. Ir. J. Weerheijm (TNO) it is concluded that schematisation of the BLEVE load to be triangular shaped is plausible.

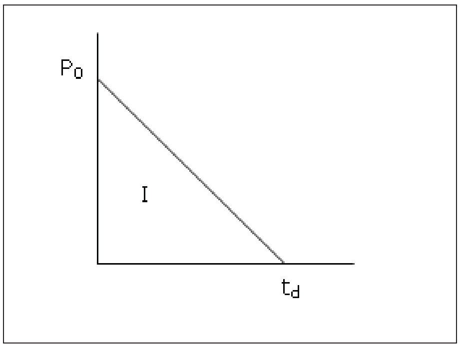


Figure 4-1 Schematisation of the load

The impulse of a triangular shaped load in time can be described as follows.

$$I = \tfrac{1}{2} \cdot P_0 \cdot t_d \tag{4.37}$$

And therefore the duration of the load can be determined as follows.

$$t_d = \frac{2 \cdot I}{P_0} \tag{4.38}$$

In January 2009, the results of this research became available, from which it can be concluded that also a BLEVE due to 50 m³ LPG with a temperature of 340 k is investigated [13]. The occurring pressure and impulse are higher than for the load case described above, which is considered to be representative in this thesis.

50 m³ LPG with a temperature of 340 k: Po = 1680 kPa and I= 63.5 kPa·s.

For this thesis the earlier described load was already used when the results of the research became available however. The results presented in this thesis are still of relevance nevertheless, though it should be noted that possibly the higher load should be considered.

The representative load as it is used in this thesis is graphically presented in the figures below.

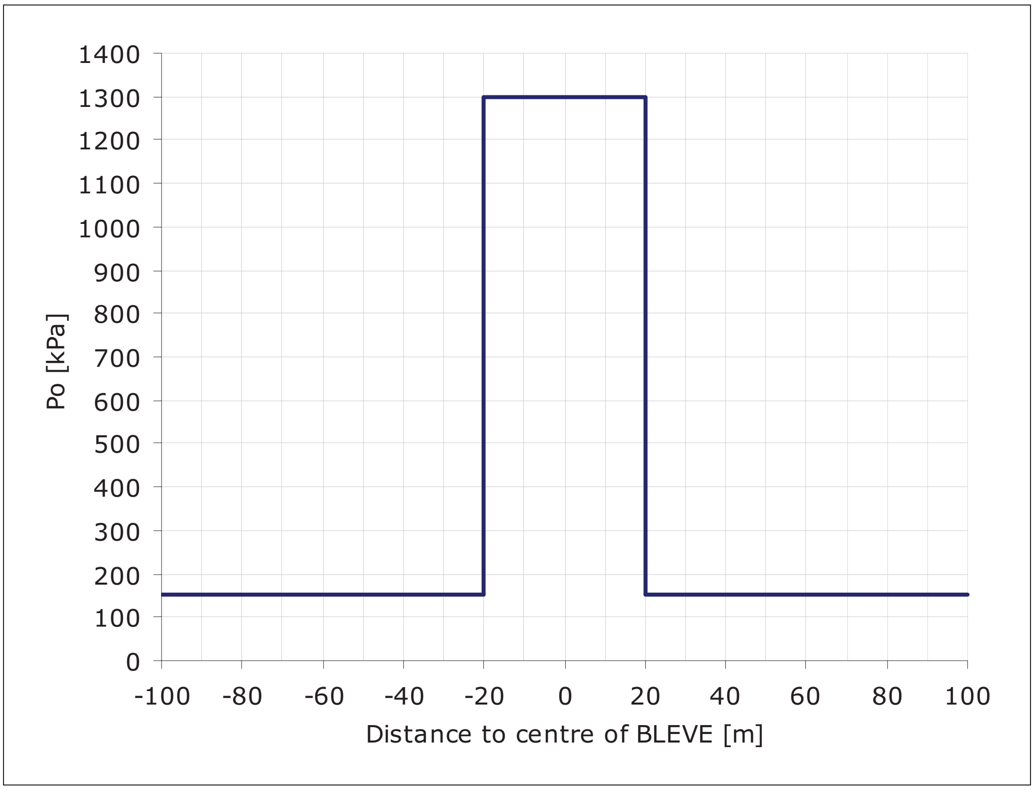


Figure 4-28 Dynamic peak pressure due to the normative BLEVE load according to TNO

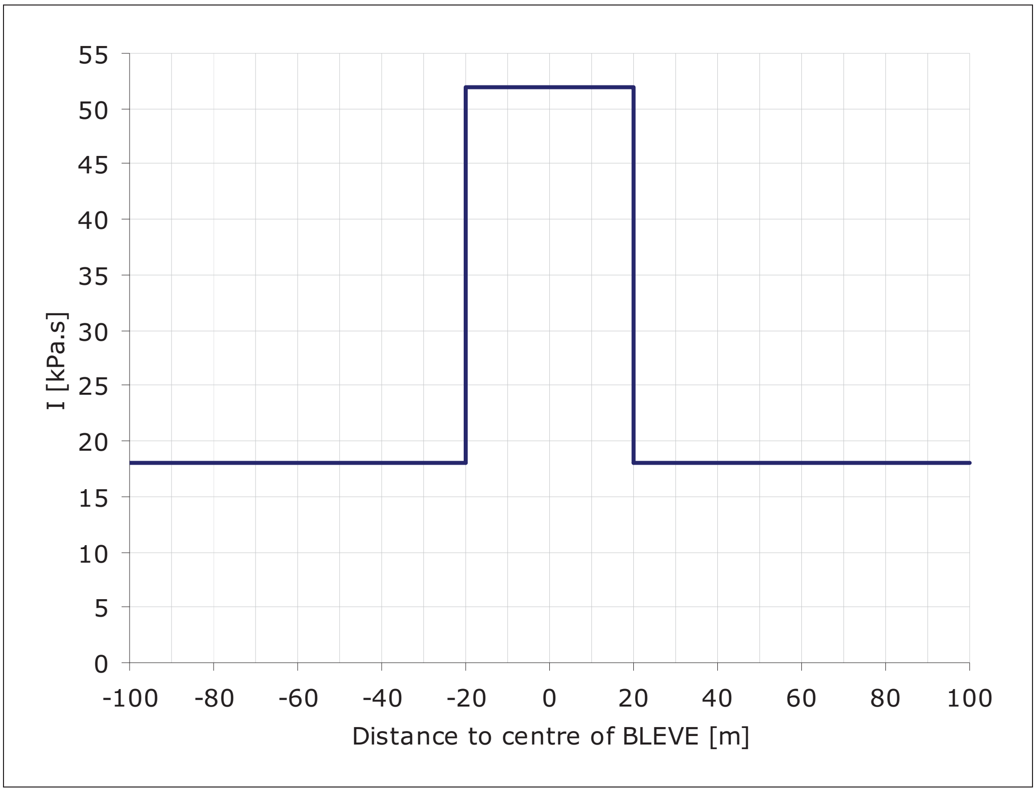


Figure 4-29 Impulse due to the normative BLEVE load according to TNO

As stated before, the load is schematized to be of triangular shape. An indication of the response can be determined using the mass spring model, as described in 4.2.2. If the order of magnitude of the response is known, it is possible to determine the order of magnitude of the statically equivalent load for the representative BLEVE load.

From the Analysis it is concluded that the dimensions of the several elements are of important influence to the response. Therefore, a representative cross-section is required. One tube will be considered to evaluate the requirement. The spans and height are estimated. Since the explosion load is of considerable magnitude, it is assumed that the thicknesses of the several elements are larger than usual. The assumed dimensions are presented in figure 4-30.

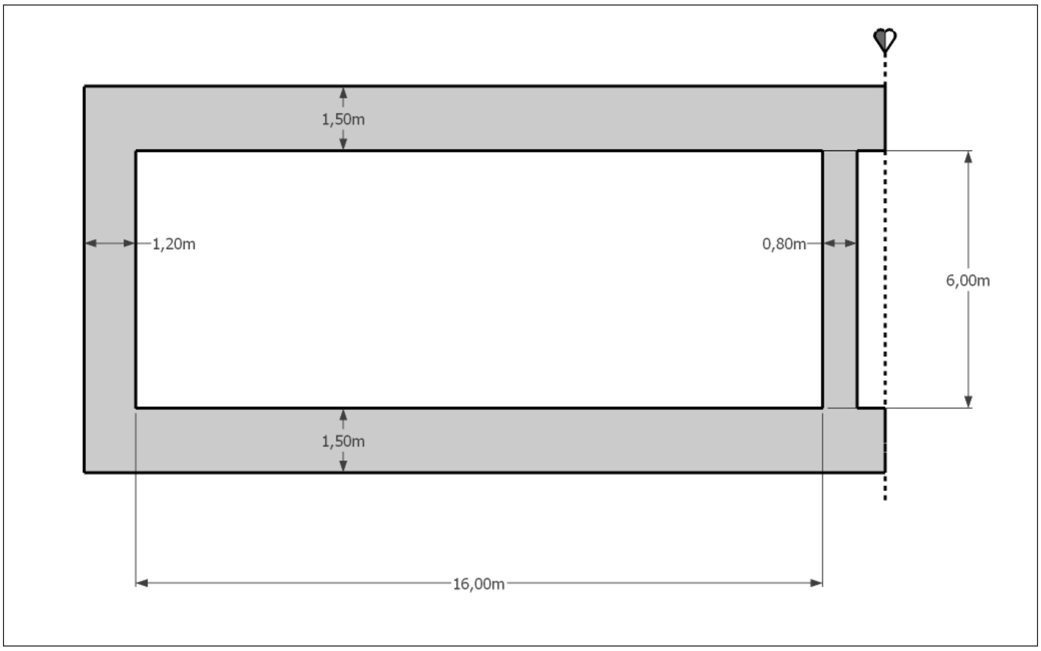


Figure 4-30 Assumed representative cross-section for the evaluation

4.5.2 Review with the mass spring model

The assumed representative structure is exposed to the representative load case according to TNO. The results of the calculation of the response for the first 20 meters, by means of the mass spring model are presented graphically in figure 4-31.

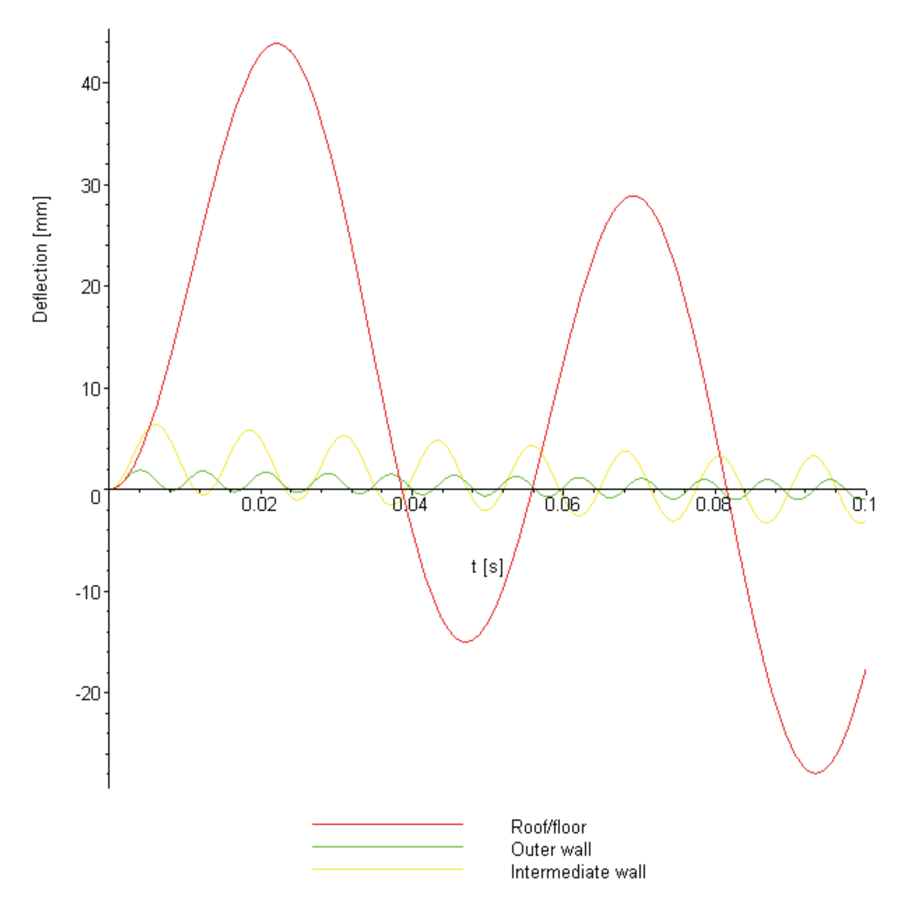


Figure 4-31 Response to the representative BLEVE load for the first 20 meters

Subsequently, the statically equivalent loads can be determined for each element. The results of these calculations are listed in the table below.

	P_0 [kPa]	I [kPa s]	Deflection according to mass spring model [mm]	$P_{Statically\ equivalent}$ [kPa]	DLF
Roof	1300	52	48↑	2452↑	1.9
			30↓	1532↓	1.2
Floor	1300	52	48↓	2452↓	1.9
			30↑	1532↑	1.2
Intermediate wall	1300	52	8→	2351→	1.8
			4←	1567←	1.2
Outer wall	1300	52	2←	2600←	2
			<1→	1300→	1

Figure 4-32 Statically equivalent loads for representative load case

- Although the schematization is rather simple, it can be concluded that the statical requirement stated for the Oosterweel tunnel does not match with the representative load case defined by TNO.
- The ratio between the outward and inward directed loads matches quite well however.

Beyond 20 meters of the centre of the BLEVE, the occurring pressure and impulse reduces largely. It will be checked if the structural capacity of the assumed cross-section is sufficient to withstand this load. The response is graphically presented in figure 4-33.

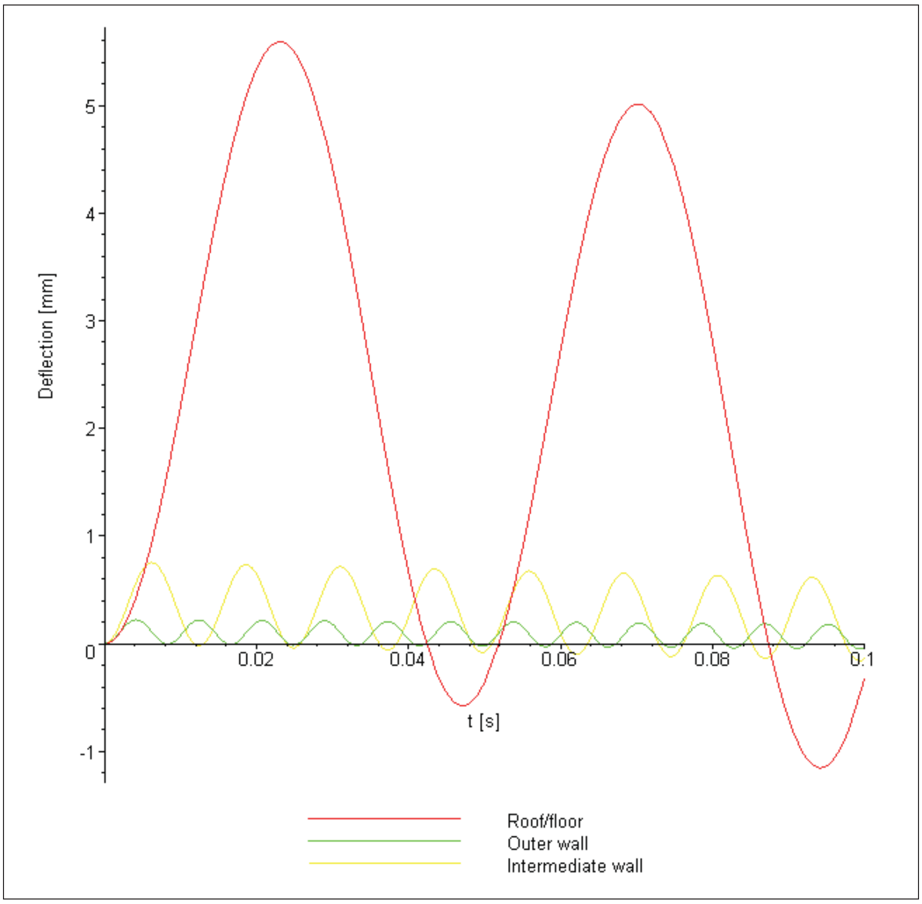


Figure 4-33 Response to the representative BLEVE load according to TNO beyond 20 meters mass spring model

	P_0 [kPa]	I [kPa s]	Deflection according to mass spring model [mm]	$P_{Statically\ equivalent}$ [kPa]	DLF
Roof	150	18	6↑	286↑	1.9
			1↓	51↓	0.3
Floor	150	18	6↓	286↓	1.9
			1↑	51↑	0.3
Intermediate wall	150	18	<1→	235→	1.6
			0←	0←	0
Outer wall	150	18	0.2←	265←	1.8
			0→	0→	0

Table 4-9 Static equivalent loads and DLF for the representative BLEVE load according to TNO

- The load that should be taken into account according to TNO beyond 20 meters from the centre of the BLEVE results in statically equivalent loads with a maximum of 286 kPa directed outward and 51 kPa directed inward.
- Beyond 20 meters from the BLEVE, the requirement of 500 kPa outward directed and 300 kPa inward directed seems to be sufficient to withstand the representative load.

The foregoing statements are graphically presented in figure 4-34. Since in the requirement for a deflagration is usually taken into account in the design of tunnels in the Netherlands this is also displayed. The structural capacity of existing tunnels is insufficient beyond 20 meters from the BLEVE and total destruction will be expected.

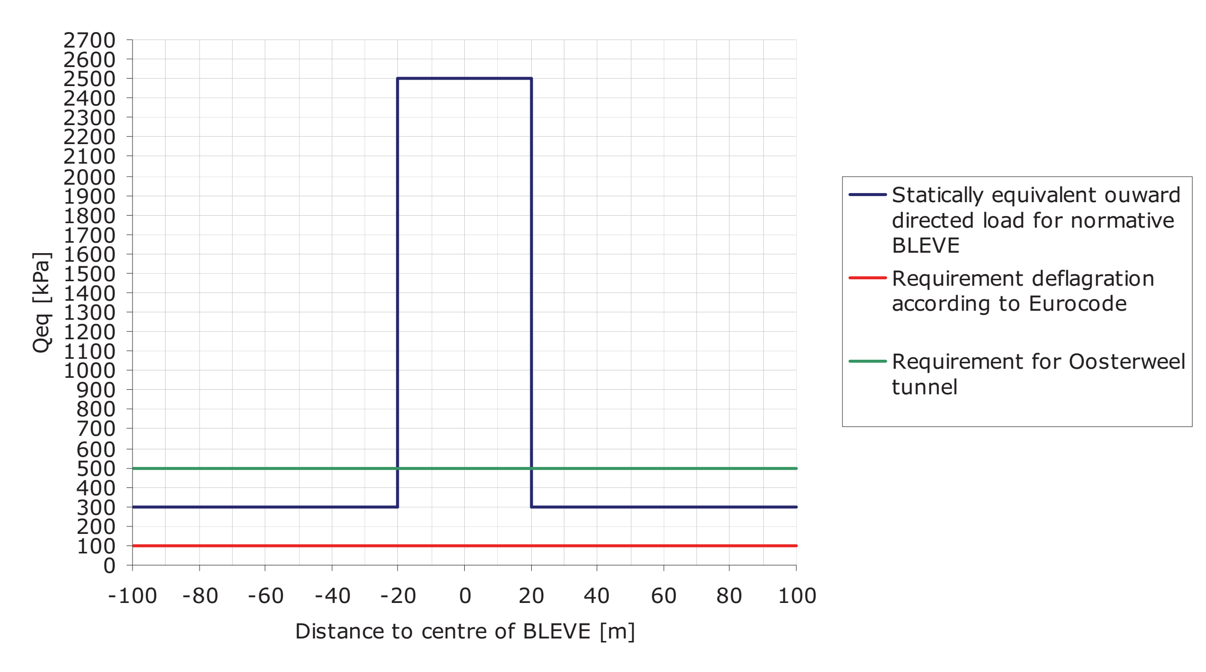


Figure 4-34 Review requirement outward directed load for the Oosterweel tunnel determined with mass spring model

The results for the inward directed load and requirements is presented in figure 4-35 According to the Euro code, no inward directed load has to be taken into account.

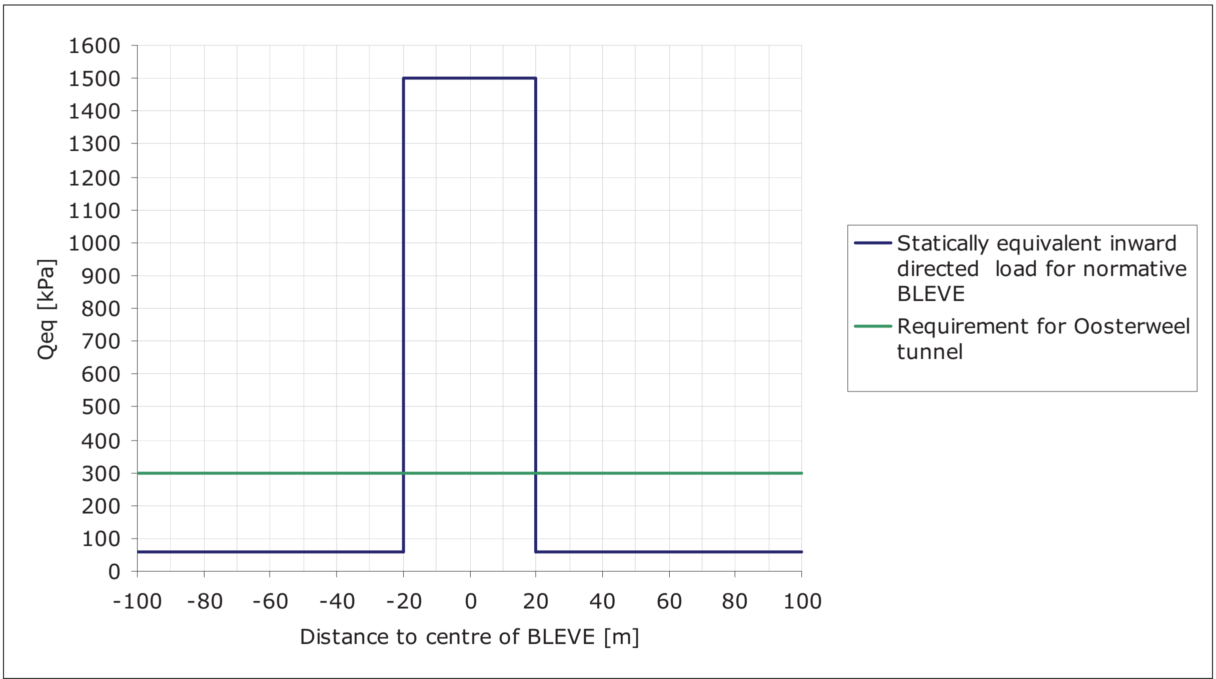


Figure 4-35 Review requirement inward directed load for the Oosterweel tunnel determined with the mass spring model

It can be concluded that the requirement stated for the Oosterweel tunnel is insufficient for the first 20 meters from the BLEVE. Beyond 20 meters, the structural capacity will be sufficient though. In existing tunnel, no inward directed load is taken into account, though the magnitude beyond 20 meters from the BLEVE is only limited and therefore the structural capacity will be sufficient probably.

Due to the severe magnitude of the load, the assumption of linear elastic material behavior is however not valid for the peak load.

According to the Euro Code, the area of reinforcement should not exceed 4 % of the cross-section. Assuming that 1.5 % of the cross-section consists of reinforcement that has a favourable influence to the resistance against explosion load. The plastic bending moment of the slab can be estimated as follows.

The maximal tensile force in the reinforcement can be described with the following relation.

$$N_{s;u} = \omega_{eff} \cdot h \cdot b \cdot f_{y;steel} \tag{4.39}$$

The height of the compression zone can be estimated as follows.

$$x_u = \frac{N_{s;u}}{b \cdot f_c^{'}} \tag{4.40}$$

$$M_p = N_{s;u} \cdot (h - c - \tfrac{1}{2} \cdot x_u) \cdot 10^{-6} \tag{4.41}$$

With the following relation, it can be determined for which load the plastic bending moment will be reached, assuming that the elements are clamped at both sides.

$$q_p = \frac{12 \cdot M_p}{l^2} \tag{4.42}$$

Where:

$N_{s;u}$	= The ultimate tensile force in the reinforcement		[N]
$f_{y;steel}$	= The tensile strength reinforcement steel,	435	[N/mm²]
x_u	= The height of the compresion zone		[mm]
$f_c^{'}$	= Compressive capacity of concrete,	21	[N/mm²]
ω	= The effective percentage of reinforcement		[-]
h	= The thickness of the element		[mm]
b	= The considered width		[mm]
c	= The concrete cover,	50	[mm]
M_p	= The plastically bending moment		[kNm/m]
q_p	= The distrubuted load that results in M_p		[kPA]
l	= The span of the element		[m]

Due to the water and soil pressure there will usually be an axial force acting on the elements, which has a favourable influence on the structural capacity, therefore the approach that is used here is a little conservative. For the elements of the representative considered cross-section the results are listed in the table below. The loads that result in exceeding the plastic bending moment are printed red.

Element	M_p [kNm]	q_p [kPa]	$q_{eq\ out}$ [kPa]	$q_{eq\ in}$ [kPa]
Roof	11900	560	2452	1532
Floor	11900	560	2452	1532
Intermediate wall	3270	1090	2351	1567
Outer wall	9540	2510	2600	1300

Table 4-10 Exceedance of plastic bending moments

It can be concluded that for the roof slab the plastic bending moment are exceeded. If the plastic deformation remains limited, it may be acceptable. For this first evaluation it is decided that the plastically bending moments should not be exceeded, in order to be at the safe side. Since the magnitude of the equivalent loads is large compared to the load that results in a plastic bending moment for the roof slab, plastic hinges will develop and failure is to be expected. For the floor a similar result is obtained, it should be noted however that this element is continuously supported by the subsoil in reality and therefore less sensitive to the occurring load. For the intermediate wall also failure is to be expected. The outer wall has almost sufficient capacity and the response of this element is therefore described rather well by the mass spring model.

From this analysis, it can be concluded that the cross-section with rather large thicknesses has insufficient structural capacity and will fail as a result of the representative BLEVE load as determined by TNO. Due to the exceeding of the plastic bending moments, the mass spring model is not valid. It can however be concluded that the requirement as stated for the Oosterweel tunnel does not represent the BLEVE load at all.

4.5.2.1 Conclusions

- In paragraph 3.2.2 it is explained that the occurrence for a gas explosion is very unlikely. It is therefore not to be expected that this is the nature of the stated requirement.
- According to TNO, 2008 the representative load for a tunnel is due to a LPG BLEVE. A peak pressure and impulse are defined.
- From this rather simple analysis it can be concluded that the requirement as stated for the Oosterweel tunnel does not comply with the representative load case as stated by TNO, May 2008.
- Since a static load of an order 5 times as large as the requirement for the Oosterweel tunnel is found with the mass spring model. The plastic bending moments of the elements are however exceeded and therefore the response it not described correctly with the mass spring system for the first 20 m from the centre of the BLEVE.
- Since the order of magnitude of the distributed loads are significantly larger than 500 kPa, it can be concluded that the representative BLEVE load according to TNO definitely can not be described equivalently by the requirement stated for the Oosterweel tunnel.
- Beyond 20 meters from the centre of the BLEVE, the requirement seems to be plausible however. The mass spring model provides a good indication here, since the plastically bending moments are not exceeded.
- Due to the severe load that occurs according to TNO in the centre of the BLEVE complete destruction is expectable locally if a regular cross-section will be applied. Since the tunnel is situated under water, this may result in collapse and inundation of the complete structure.

4.5.3 Review with the dynamic module of Plaxis

In order to investigate the effect of the load to the representative cross-section, it is decided to make use of the finite element code Plaxis. With this software package it is possible to model the structure and surrounding soil. For the structure it is possible to perform plastic analysis. The dynamic module of the program enables the application of a load that varies in time. It is possible to calculate the occurring displacements and forces.

In order to investigate the response of the tunnel to the explosion load, a sufficiently large area should be considered. Since a dynamic problem will be considered, the model will be provided with absorbing boundaries in order to prevent the results to be influenced by reflections. For simplicity, the soil is schematised to consist of Pleistocene sand. On top of the tunnel, 15 meters of water is assumed to be present. The representative cross-section as indicated in figure 4-30 is modelled.

An indication of the schematisation is depicted in figure 4-36.

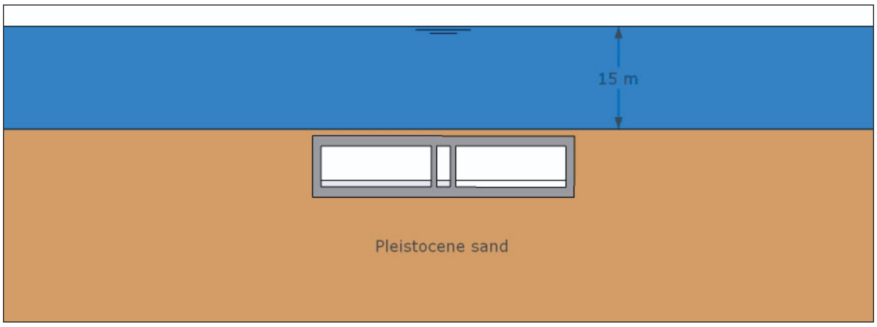


Figure 4-36 Model as used for Plaxis

The HS small material model is used to model the soil, since this material model is very suitable for dynamic problems. The following parameters were used for the schematisation of the Pleistocene sand.

γ_{unsat}	17	[kN/m³]
γ_{sat}	20	[kN/m³]
E_{50}^{ref}	$4 \cdot 10^4$	[kN/m²]
E_{oed}^{ref}	$4 \cdot 10^4$	[kN/m²]
E_{ur}^{ref}	$1.2 \cdot 10^5$	[kN/m²]
Power	0.7	[-]
C_{ref}	1	[kN/m²]
φ	31	o
ψ	0	o
$\gamma_{0.7}$	$1 \cdot 10^{-4}$	
G_0^{ref}	$1.5 \cdot 10^5$	[kN/m²]
R_{int}	0.9	-
α	0.001	
β	0.0015	

Tabel 4-1 Parameters Pleistocene sand

The concrete structure is composed of several elements and all connections are assumed to be fixed. Elasto plastic material behaviour is assumed and estimation for the plastically bending moment and axial force are made in correspondence to paragraph 4.5.1. The combination of the bending moment and axial force is determines if elastic or plastic deformation will occur. In Plaxis this is modelled as indicated in the figure below. Within the shaded area elastic material behaviour occurs, whereas outside this area the material behaviour will be plastic.

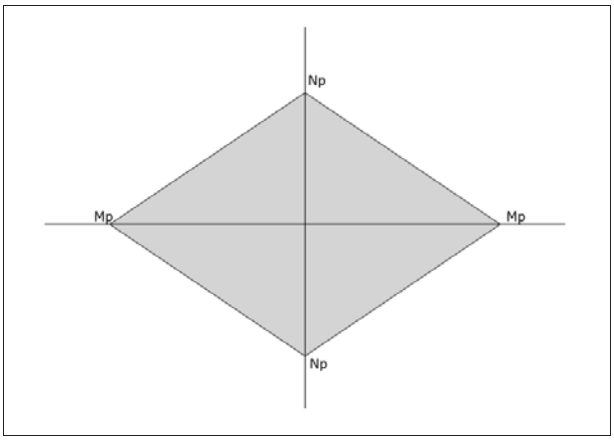


Figure 4-37 Material model Plaxis

In order to give a more accurate judgement with respect to the response of the representative cross-section, the surrounding soil as well as the water on top will be taken into account.

The occurring deformations that are acquired from the dynamic analysis are presented in figure 4-38 below. The deformations are scaled up three times in this picture, for clarity.

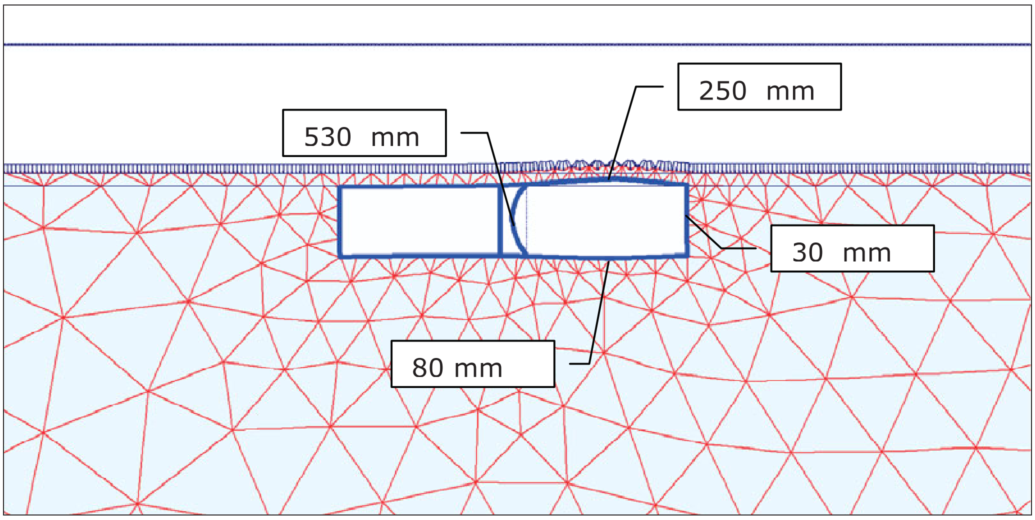


Figure 4-38 Deformations at 0.1 s calculated with Plaxis, scaled up 3 times

It can be concluded that the intermediate wall and roof slab have insufficient structural capacity and failure is expected. In order to solve the problem, the thicknesses should be increased. The required dimensions can be estimated by means of trial and error whereby it is continuously checked if the plastically bending moment is not exceeded. The results of the iteration are listed in table 4-11. The focus of this analysis is on the structural capacity, for now the buoyancy of the element will not be considered.

Element	Thickness [mm]
Roof	3100
Floor	1800
Int. wall	1300
Outer wall	1200

Table 4-11 Estimated required thicknesses

Since the outer wall and floor slab are less vulnerable, the thicknesses of these elements will be increased slightly. For the roof slab a thickness of 1.8 meter and for the intermediate wall a thickness of 1.3 m will be used. The estimated dimensions were implemented in the Plaxis model and the calculations are made again. The displacements for the points indicated in figure 4-39 are presented in figure 4-40.

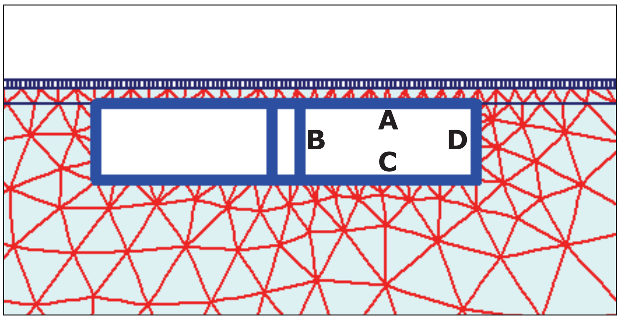


Figure 4-39 Considered locations

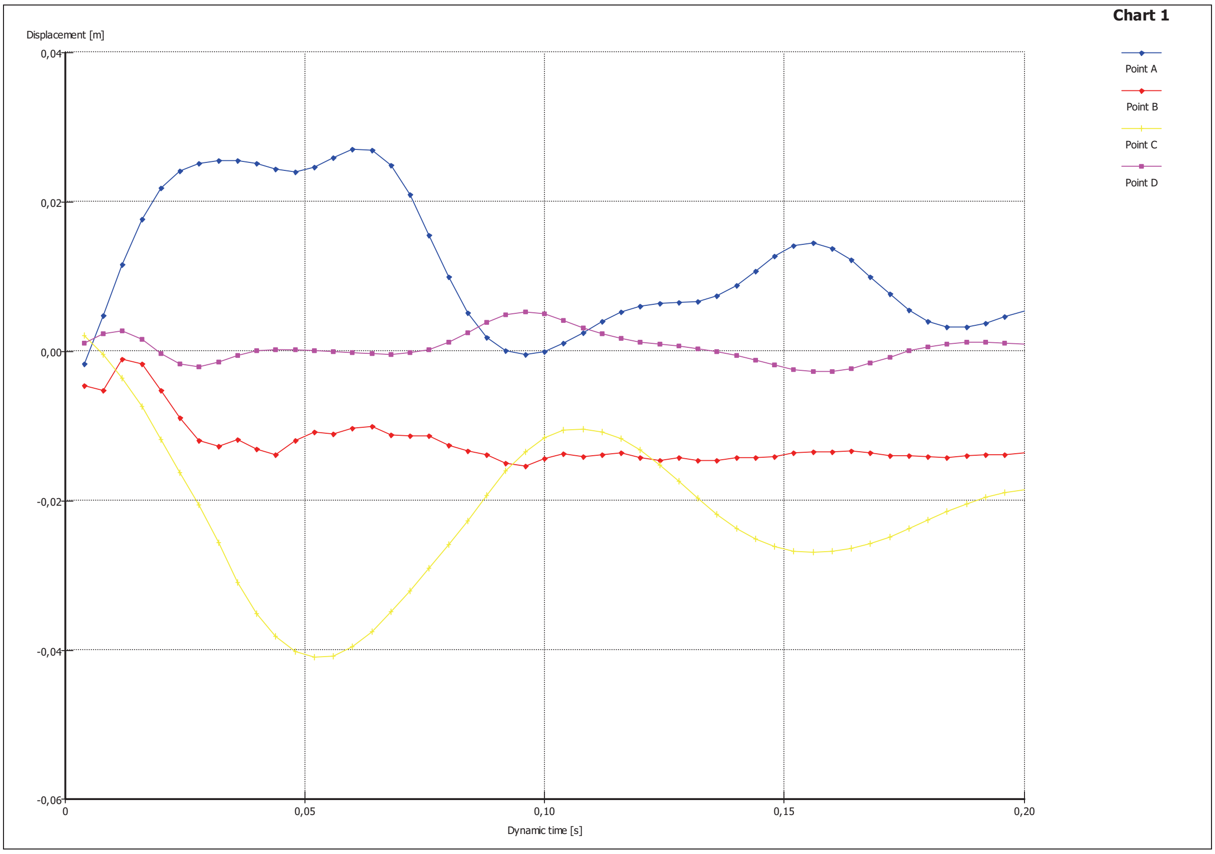


Figure 4-40 Displacements adapted cross-section

It can be concluded that the displacements are reduced significantly as a result of the adaptations. The deformations for the adapted cross-section are of acceptable magnitude. Optimization could be performed, though the order of magnitude for the required dimensions is clear. These dimensions are excessive as indicated in figure 4-41.

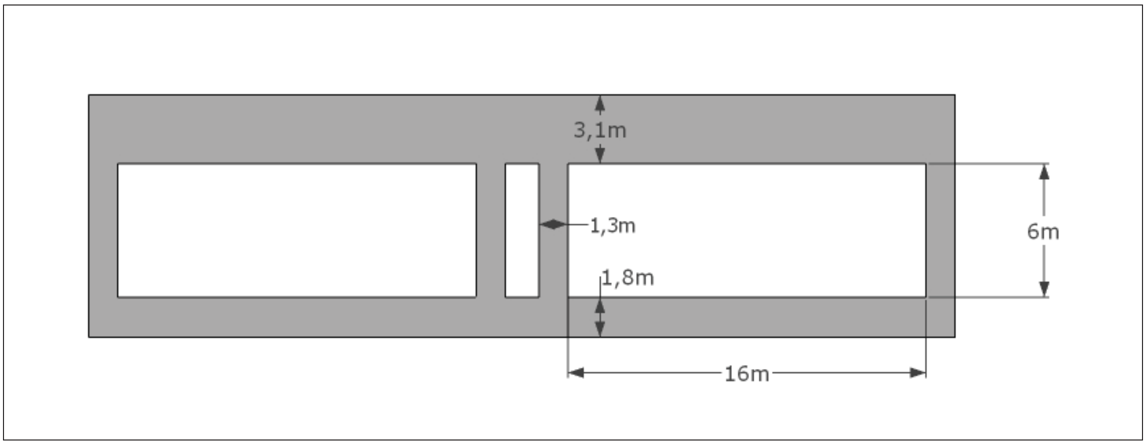


Figure 4-41 Adapted cross-section

The cross-section as it is considered now will not float by far. Furthermore, the cross-section cannot be adapted to have sufficient buoyancy in an efficient way. Therefore, this is not a solution to the problem. It is however illustrative for the effect of the representative BLEVE load according to TNO on the design of the cross-section.

5 Introduction to the structural feasibility study

This chapter provides relevant information concerning the backgrounds and starting points that will be used for the feasibility study to the design of an explosion resistant immersed tunnel. A cross-section of a recent project will be used for this study. Although the requirement for the Oosterweel tunnel does not describe the representative load, the effect of this load to the design will be investigated.

5.1 *General*

In order to explore the possibilities for the design of an explosion resistant immersed tunnel, it is decided to consider a representative cross-section. Recently, for tunnel projects it is required that relatively high explosion loads have to be withstood. There seems to be a trend to do so in Europe, therefore it is useful to consider these extreme loads and the connected consequences for the design and construction. In this respect, two recent tunnel projects are considered. First, the Oosterweel tunnel, which is planned to be constructed in the city of Antwerp, Belgium. One of the requirements for this project concerns the capacity to withstand a high overpressure as a result of an explosion. Due to a number of other requirements, the cross-section of this tunnel is rather unique and therefore the project is a more or less individual case as discussed in paragraph 2.4. It is decided to perform a more general study and increase the applicability for the Dutch situation, with an eye to the future. To this end another recent tunnel project is considered, the second Coen tunnel located near Amsterdam, the Netherlands. The functional requirements for this project are considered to be representative for the present and near future. The second Coentunnel will be used for the case study, whereas the requirement for explosions will be adopted from the Oosterweel tunnel. The following paragraphs provide information about the second Coentunnel and a detailed description of the starting points and cross-section will be provided subsequently.

5.1.1 Objective

The purpose of the study is to explore the technical feasibility of structural measures in order to provide capacity to withstand an explosion load.

5.1.2 Structure of the elaboration

At first, the design of the tunnel in case there are no requirements concerning the explosion load will be considered. In this way more insight is provided in de adaptations and extra provisions that are required compared to a regular tunnel. Secondly, a number of alternatives will be developed in order to find a realistic and applicable solution. First of all the principle of a solution will be discussed. The most promising solutions will be selected for further elaboration. Subsequently a exploring calculations will be made for the particular alternatives. Besides this, the execution phase will be considered. Based on a multi criteria evaluation, the alternatives can be ranked. Promising solutions will be considered in more detail and finally a judgement can be made if it is feasible to build an immersed tunnel with these extraordinary specifications.

5.1.3 Stuctural design

A strong interaction between the execution phase, transport phase and final situation is characteristic for an immersed tunnel. Actually, for the different stages contrary demands hold. During transportation the elements should be afloat, whereas in the final situation it should be ensured that the elements remain stable at the bed. Therefore, usually the elements are designed in such a way that these are just afloat during transport. Immersion then takes place by filling water tanks that are placed inside the element. After the immersion process this water ballast is replaced by a layer of non reinforced concrete. For regular projects, no high explosion load has to be taken into account. The dimensions of the slabs and plates are determined in such a way that the element has a freeboard of approximately 1 % only. Requirements concerning strength or stiffness are hardly ever normative for the thickness of the walls and slabs. In case a high explosion load will be taken into account, it may however occur that the structural requirements become normative instead.

The design process is schematically presented in the figure below.

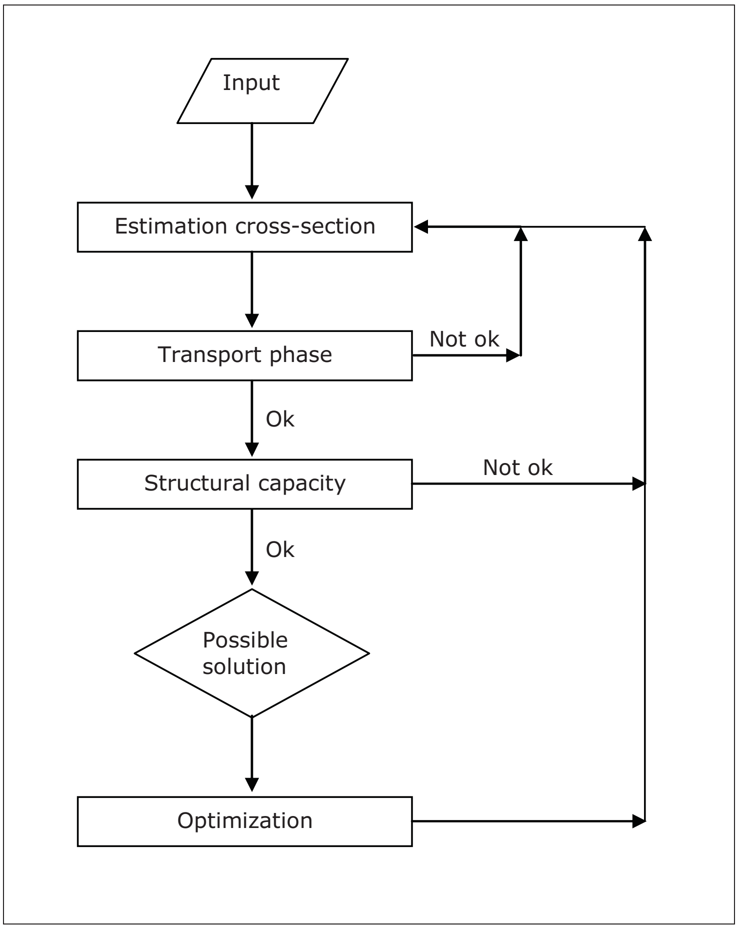


Figure 5-1 Flow chart structural design

The input for the design process consists of requirements, starting points, boundary conditions and data concerning materials and the surroundings for example. The dimensions of the cross-section for the second Coen tunnel as developed by BAM Infraconsult will be used as a starting point. Since an explosion load was not taken into account for the development of this cross-section, the dimensions probably have to be adjusted. An extra complication is that the elements should be transported afloat. The adjusted cross-section should comply with this requirement. If both the structural and transport requirements are met, a possible solution is found. In order to find a more optimal solution the process can be repeated.

It can be concluded that there exists a very delicate balance between the requirements concerning transport and the final stability that should be monitored and adjusted continuously during the design phase. To this end, a spreadsheet with which the stability during transport can be checked is developed.

5.2 *The second Coentunnel*

Amsterdam is the capital of the Netherlands. A High concentration of economical activities is found in its vicinity. The city has approximately 750.000 inhabitants, while 1 million people live in its region.

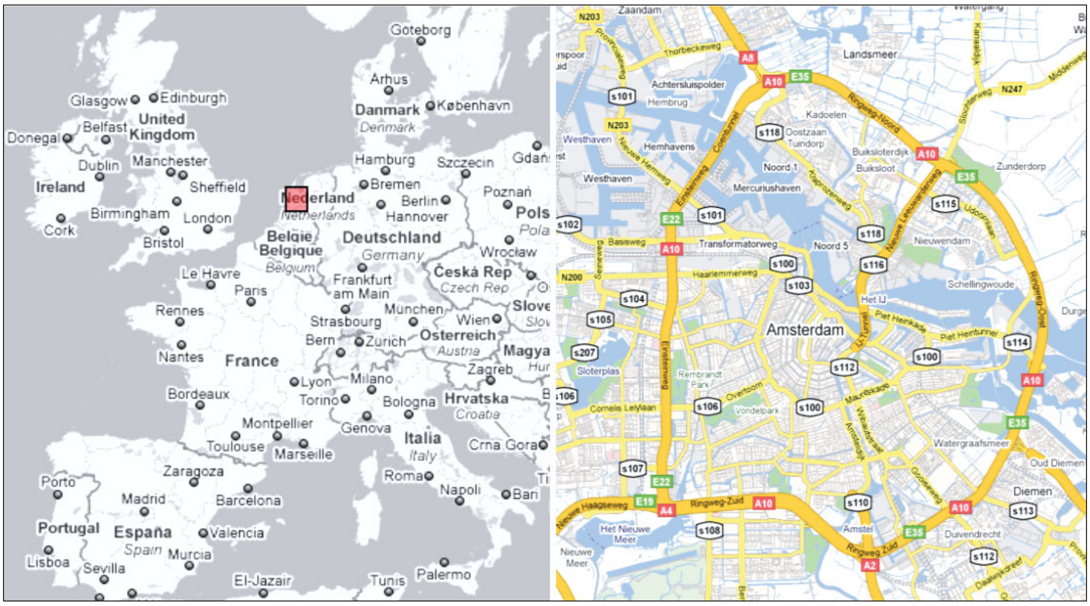


Figure 5-2 Location of Amsterdam

In the densely populated area, a lot of road traffic is daily present. The capacity of the existing network is however insufficient, resulting in congestion during the rush hours. One of the major bottlenecks is the existing Coentunnel, which crosses the North Sea channel. In the current situation, daily 110.000 vehicles make use of the tunnel, which has only two lanes in each direction. Therefore, it is decided to build the second Coentunnel, next to the existing one. The target is to reduce the daily congestion problems on the A8 and A10 west, in order to improve the connectivity of the city from the north. The construction of the tunnel will result in the transfer of the congestion towards the A10 west, part of the ring road of Amsterdam. Therefore, also the realisation of the Westrand road (A5) is planned. This road connects the Coentunnel and the node Raasdorp, the dashed line in figure 5-3 indicates this trace. Besides this, the A8 and A10 will be extended and widened to realize a better flow. The available budget for the project is approximately € 1.1 billion. After the tender phase, the client, Rijkswatersaat, awarded the project to Coentunnel Company BV, a combination established by 7 companies. The construction works will start in 2009.

Figure 5-3 Trace Westrand road, Amsterdam

The second Coen tunnel should provide five lanes in total. Together with the existing Coentunnel 8 lanes will be present. One of the criteria for the new tunnel is that two of these lanes should be usable in both directions, in order to be able to anticipate better to the need for capacity during the rush hours. The second Coentunnel should therefore exist of one tube that provides space for three lanes and one tube that provides 2 lanes, the future situation is depicted in figure 5-4 .The cross-section complies with the requirements that are representative for the present and near future and is therefore very suitable to use for this research.

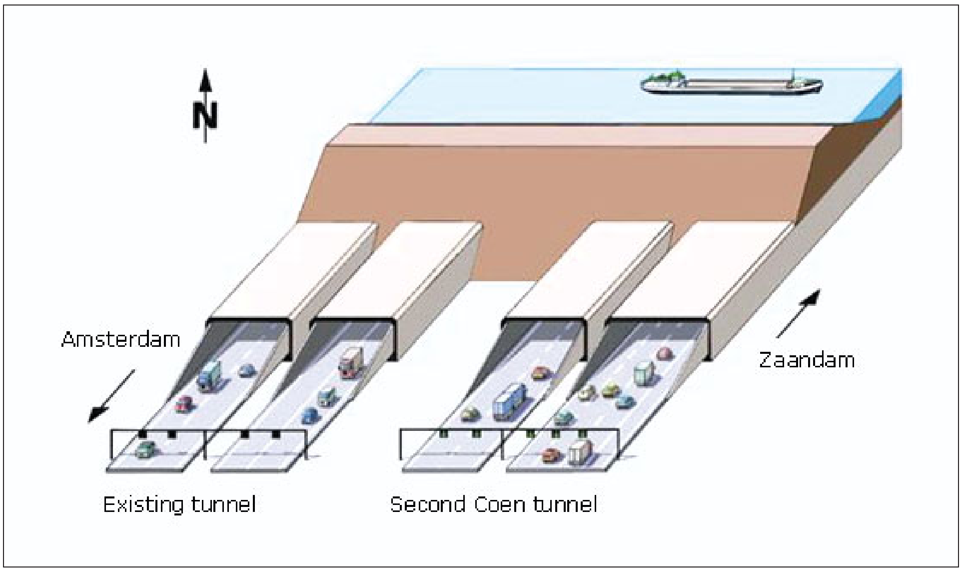


Figure 5-4 Desired situation Coen tunnel

For the technical feasibility study, it is decided to consider a cross-section that general enough to make the results of the research applicable for future projects. To this end, it is decided to focus on the second Coen tunnel. The alignment and cross-section from the design for the second Coentunnel, made by BAM Infraconsult will be used as a starting point.

5.2.1 Cross-section

The cross-section developed by BAM Infraconsult in the tender phase for the second Coentunnel will be used as a starting point to this end. This cross-section is presented in the figure below.

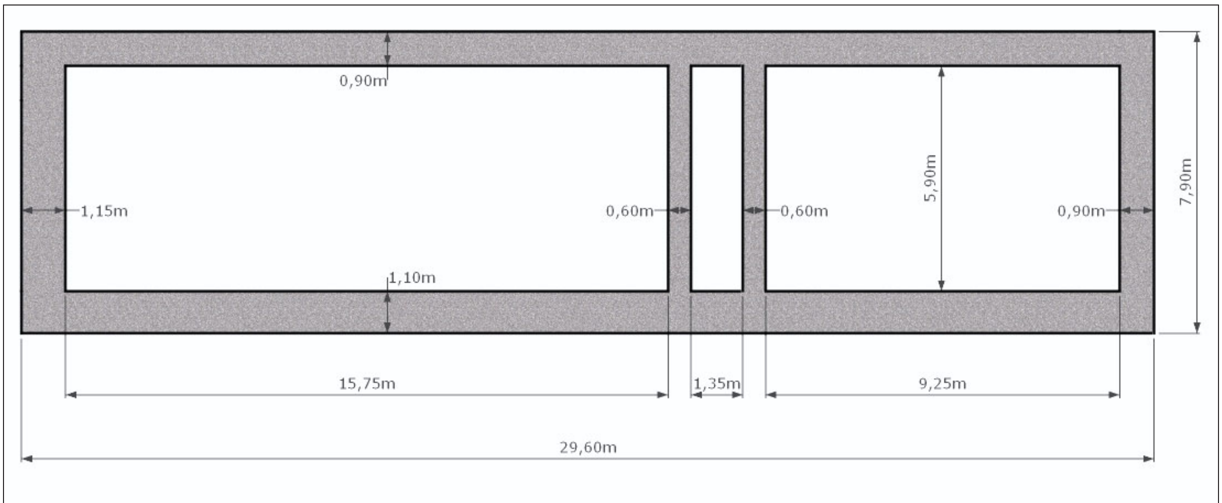


Figure 5-5 Cross-section second Coentunel, developed by BAM Infraconsult

5.2.2 Vertical stability

In order to check if the element complies with the stability requirements for the transport phase, a spreadsheet is developed, the lay out is presented in Appendix B. All spans and thicknesses can be varied whereas the draught of the element is calculated. A few important assumptions and remarks connected to these calculations are listed below.

- In order to be on the safe side, different unit weights for the concrete and water are used for the transport phase and final situation. For the transport phase a certain freeboard is required, therefore a low density of the water and a high unit weight for the concrete are unfavourable. In the final situation, vertical stability should be ensured, therefore a high density of the water and a low unit weight for the concrete are normative in that stage. The values that are listed in table 5-1 are considered to be less favourable for the particular situations.

Stage	Reinforced Concrete	Water
Transport phase	25 kN/m³	10 kN/m³
Final situation	24 kN/m³	10.35 kN/m³

Table 5-1 Unit weights

- At the connection between the walls and the roof slab, it is common to apply larger thicknesses locally. For these calculations, the extra amount of concrete is not taken into account.

- In [22] estimation for the deadweight of the immersion equipment and bulkheads is given. This value is assumed representative for the calculations.

- The elements for the second Coen tunnel will be constructed in a building dock near Barendrecht and transport will take place via the North Sea. To this end, a larger freeboard should be ensured, due to the waves. A minimum of 0.5 m is considered representative, since this was the requirement for comparably recent projects.

- A picture of the cross-section is generated with the spreadsheet, the water level during transport is indicated.

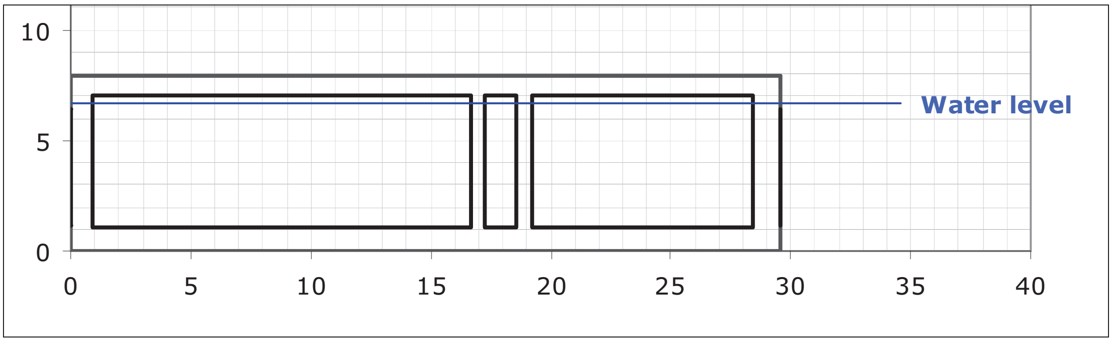


Figure 5-6 Cross-section during transport

From this simple check, it is concluded that the freeboard is 1.15 m, which is rather large.

- The inner height of the element will be partly used to accommodate ballast concrete in order to provide sufficient stability in the final situation. For this element a safety of 1.14 is found, this is sufficient, applying slightly extra ballast and providing the elements with ears will increase the stability to a larger extent.

5.2.3 Longitudinal section

The depth of the tunnel varies with the longitudinal profile, the surcharge load provides resistance against the explosion and it is likely that a location near the banks is exposed to a slightly higher surcharge load. The effect is considered to be only limited though. For simplicity, it is decided to consider the deepest cross-section that is located under the channel, indicated with red in figure 5-7.

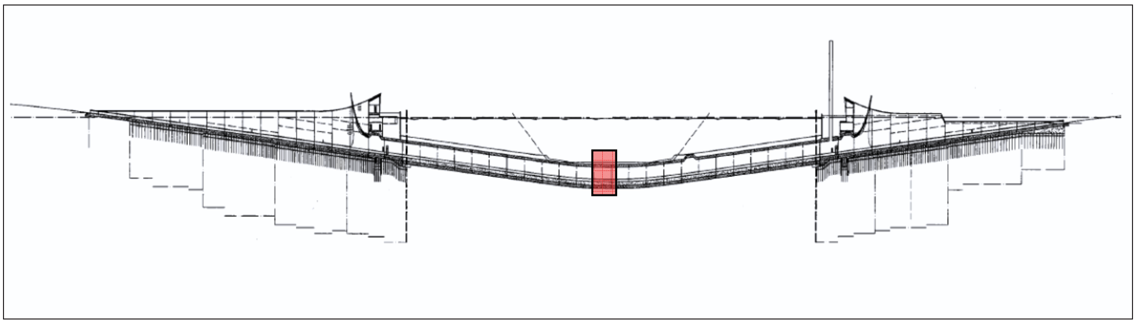


Figure 5-7 Longitudinal section second Coentunnel

For the evaluation, a water depth of 15 m is considered to be representative. Furthermore on top of the element a one meter thick layer of soil is assumed to be present. In figure 5-8 an impression of the deepest cross-section is depicted.

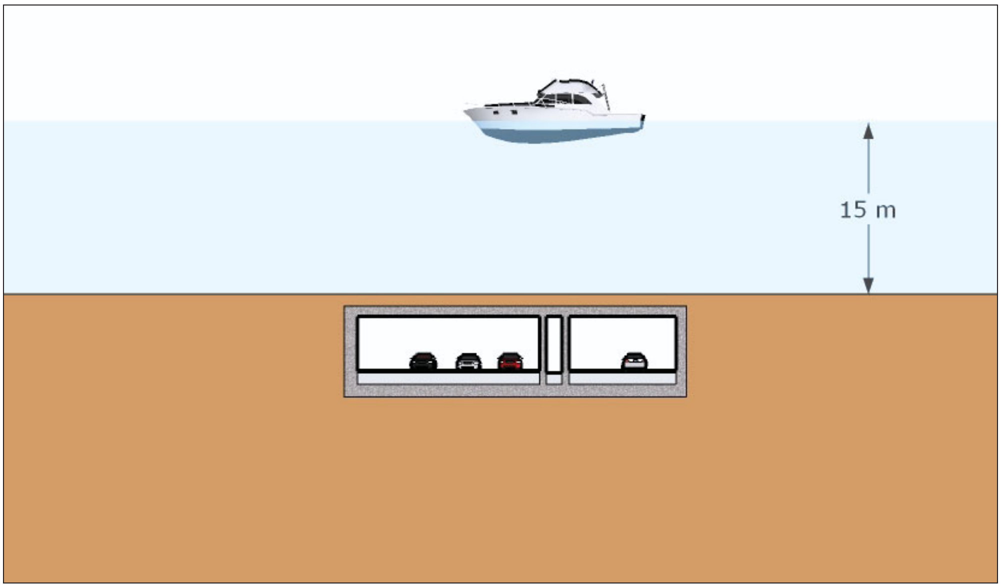


Figure 5-8 Impression of the deepest cross-section

The tunnel is completely embedded in the subsoil. The soil that is present at the sides will be taken into account as a load. The structure will be schematized to be supported by a bed of springs, of which the stiffness is described with the bedding constant as indicated in the figure below.

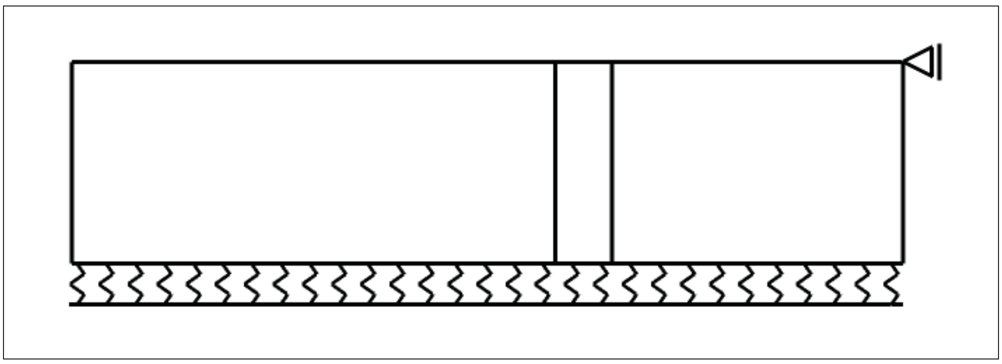


Figure 5-9 Schematisation of the cross-section

The bedding constant depends on the characteristics of the subsoil. For this analysis it is assumed that the subsoil consists of loose sand. A suitable approximation for the bedding constant is obtained from [10].

$$k' \approx 0.31 \cdot E_{soil} \tag{5.1}$$

Where

k'	= The bedding constant for a line load	[MN/m^2]
E_{soil}	= The modulus of elasticity of the soil	[MPa]

For loose sand a modulus of elasticity of 25 MPa is representative. This results in a bedding constant of approximately 8 MN/m^2.

Since the model requires a fixation in horizontal direction, in order to make calculations, a vertical sliding support is added.

5.2.4 Loads

A number of loads and load combinations should be considered in the design of an immersed tunnel. In the final situation several loads act on the element, the most important are listed below.

Permanent

- Water pressure
- Soil pressure
- Deadweight

Special loads

- Dragging anchor
- Immersing ship
- Collisions
- Fire load
- Explosion load

For this exploring study, the special loads, apart from the explosion are of less importance. Therefore, only the permanent loads will be taken into account.

Water pressure

The load that acts on the structure as a result of the water pressure is depicted schematically in the figure below.

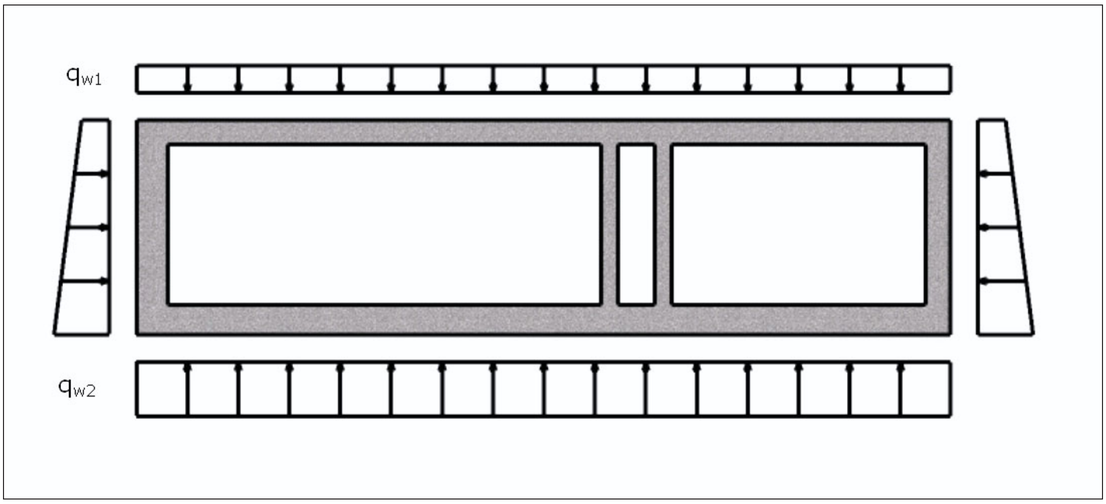


Figure 5-10 Water pressure

The following relations hold.

$$q_{w1} = h \cdot \rho_{water} \tag{5.2}$$
$$q_{w2} = (h + h_{tunnel}) \cdot \rho_{water} \tag{5.3}$$

Where

h	= Distance between the top of the roof and the watertable	[m]
ρ_{water}	= Volumetric weight water	[kN/m^3]
h_{tunnel}	= Height of the tunnel	[m]

Soil pressure
The soil pressure can be schematized as depicted in the figure below.

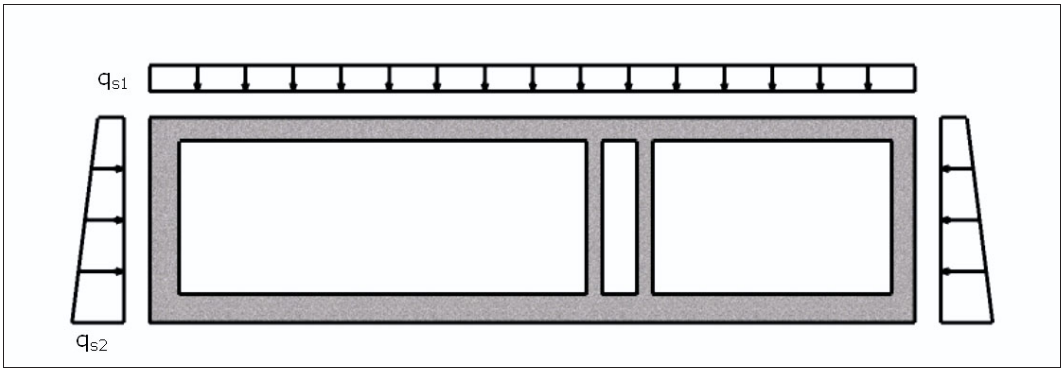


Figure 5-11 Soil pressure

The following relations apply.

$$q_{s1} = \gamma'_{soil} \cdot d \tag{5.4}$$
$$q_{s2} = \gamma'_{soil} \cdot d + k_0 \cdot \gamma'_{soil} \cdot h_{tunnel} \tag{5.5}$$

Where

γ'_{soil}	= The effective volumetric weight of the soil	[kN/m^3]
d	= The thickness of soil layer on top of the tunnel	[m]
k_0	= The neutral soil pressure coefficient	[-]
h_{tunnel}	= The height of the tunnel	[m]

Explosion
In correspondence with the requirements for the Oosterweel tunnel, the explosion will be schematized by means of a static pressure and suction. An outward directed pressure of 500 kPa and an inward directed pressure of 300 kPa should be taken into account. Since the dimensions of both tubes differ, separated load cases will be considered as indicated in figure 5-12 and figure 5-13.

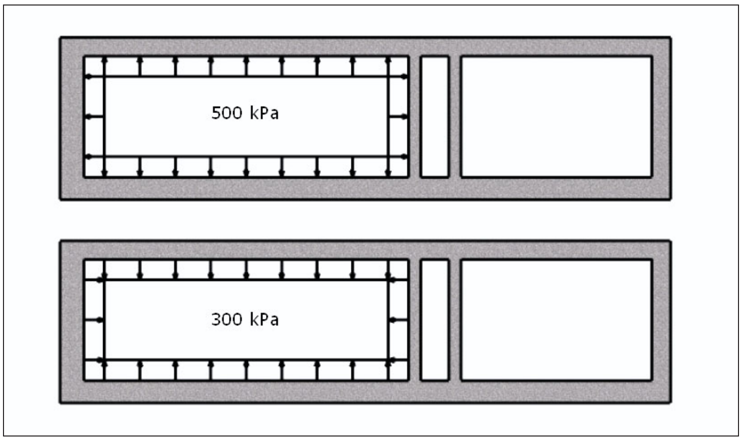


Figure 5-12 Explosion load large tube

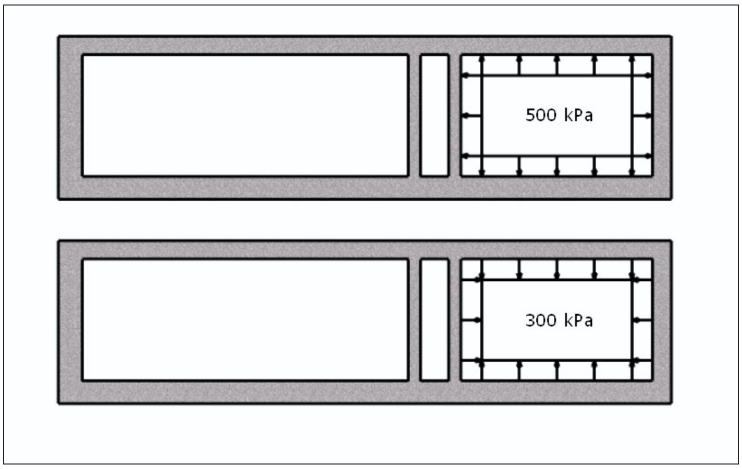


Figure 5-13 Explosion load small tube

Safety factors
According to the Euro Code, a load factor of 1.35 should be used for the permanent loads in the ultimate limit state. For the serviceability limit state a load factor of 1.0 should be applied. For incidental loads, like an explosion, a load factor of 1.0 is prescribed.

Environment
Since the tunnel is submerged under water permanently, according to the Euro Code environmental class XC1 applies.

5.3 *Design without explosion load*

The situation without taking an explosion load into account will be considered. Subsequently, the explosion load will be accounted for as well. To this end, static pressures of varying magnitude will be considered in order to determine the influence of this phenomenon.

The software package ESA PT is used to perform calculations and determine the normative bending moments, shear and axial forces. The calculations are made per meter width.

For the determination of the required reinforcement, the occurring bending moments and forces in the clear of supports are used for this purpose instead of the values at the intersection of the system lines. In case the values corresponding to the system lines would be used, unnecessary conservative results will be acquired. At the support, the occurring bending moments can be withstood easily. The principle of the choice is presented for the bending moments in the roof slab in the figure below, where A corresponds to the bending moment at the intersection of the system lines, whereas B is the value in the clear.

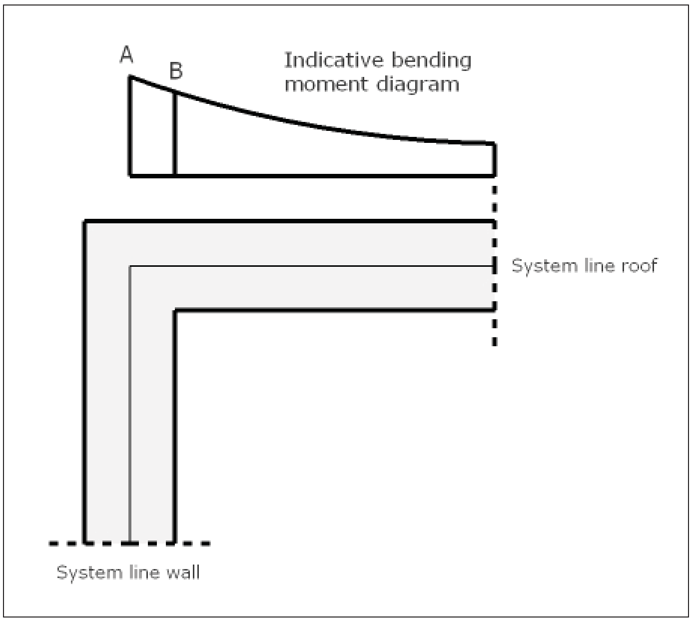


Figure 5-14 Explanation clear of the support

Subsequently, a spreadsheet is developed in order to calculate the required amount of reinforcement according to the Euro Code. For each element, the required amount of bending reinforcement as well as shear reinforcement is determined for several locations. Furthermore, it is checked if the considered section complies with the requirements concerning crack widths. In order to judge the workability, the required amount of reinforcement steel is also listed for the different locations. An extensive report with the results of the calculation is available. A sample is attached in Appendix D, a detailed overview of the required amount of reinforcement can be found in Appendix E.

The most important results are summarized in table 5-2. The total amount of reinforcement steel as well as the required amount of concrete per meter are listed in table

Concrete	79	[m^3]
Reinforcement	10000	[kg]
Overall ratio	127	[kg/m^3]

Table 5-2 Quantities per meter

5.4 *Design with explosion load as stated for the Oosterweel tunnel*

The design calculations will be made again, taking into account a static load, representing the explosion. The approach will be similar to the design without taking into account the explosion load. Since the load is quite severe, also a smaller magnitude will be considered, scenario 1.

Scenario	Outward directed load [kPa]	Inward directed load [kPa]
1	300	180
2	500	300

Table 5-3 Scenarios static explosion load

Scenario 2 corresponds to the requirement that was stated for the Oosterweel tunnel. The explosion is classified as an accidental load case. It is therefore justified to apply a value of 1.0 for the load factor. The magnitude of the explosion load is very large. In the outward direction the load corresponds to a water pressure of 30 meters whereas the inward directed load corresponds to a pressure of 18 m water. The permanent surcharge load consists of 15 meter of water and 1 meter of soil only.

In the ultimate limit state the permanent loads should be taken into account with a safety factor of 1.35. Due to the magnitude of the explosion load it is justified to make the calculations taking into account the explosion and permanent loads in the serviceability limit state only, with all load factors are equal to 1.0. For scenario 2 the effect is even more obvious.

Load case	ULS		SLS + explosion load	
	Load [m water]	Safety factor	Load [m water]	Safety factor
Inward load scenario 1 roof	16	1.35	16+18=34	1.0
Outward load scenario 1 roof	-	-	16+30=46	1.0
Inward load scenario 2 floor	16 + 8 = 24	1.35	16+8+18=42	1.0
Outward load scenario 2 floor	-	-	16+30=46	1.0

Table 5-4 Safety factors ULS and SLS with explosion load

Since this holds for the roof and floor slab, it also applies for the walls. In the ULS, there are no outward directed loads, so for these the serviceability state in combination with the explosion load is normative.

Crack width
The requirements concerning crack widths should be checked in the serviceability limit state. Since no accidental loads have to be taken into account for this check, the occurring steel stresses are expected to be very small. The occurring loads due to the accidental explosion load are in the order of twice as large as the permanent loads after all. Therefore, it is decided that crack width control is not necessary, certainly not in this stage.

Adaptations of the cross-section
It is preferable to remain the initially assumed cross-section unchanged as much as possible. The reason for this is that it is optimized for the several stages of execution and provides an efficient solution. If it turns out to be impossible to comply with the requirements, it is inevitable to make adjustments however.

Calculations
Since a number of aspects are different compared to the situation no explosion load was taken into account, also the spread sheet that is used to determine the required amounts of reinforcement is different. The structure and lay out of the spreadsheet is presented in Appendix F. The considerations and detailed results of the calculations are presented in Appendix H.
The most important results of the calculations are presented in the table below.

	Scenario 1	Scenario 2	Without explosion	
Concrete	79	110	79	[m³/m]
Reinforcement	15298	18613	10000	[kg/m]
Overall ratio	194	169	127	[kg/m³]

Table 5-5 Required quantaties for the considered scenarios

Distribution reinforcement is not included in the determined amount of reinforcement.

5.5 *Effect of explosion load on the design*

From previous paragraphs it can be concluded that the required amount of concrete and reinforcement steel increase significantly as a result of the explosion load. Figure 5-15 gives a clear indication of the magnitude.

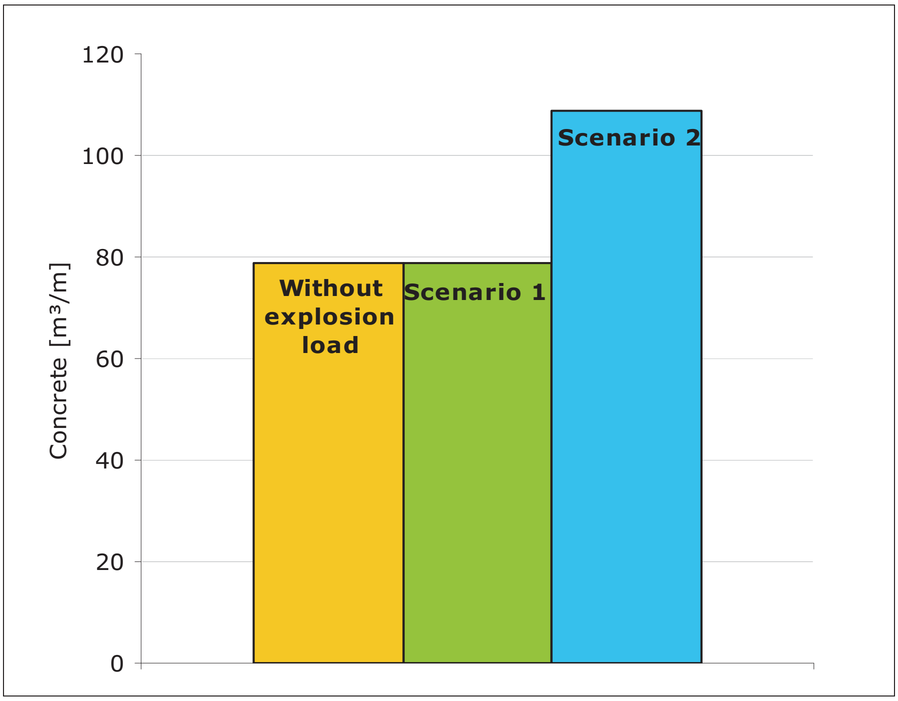


Figure 5-15 Required amount of concrete for different scenarios

The cross-section is just of sufficient dimension to comply with scenario 1, therefore no additional concrete had to be applied. For scenario 2 a significant amount of additional concrete is required to provide sufficient capacity.

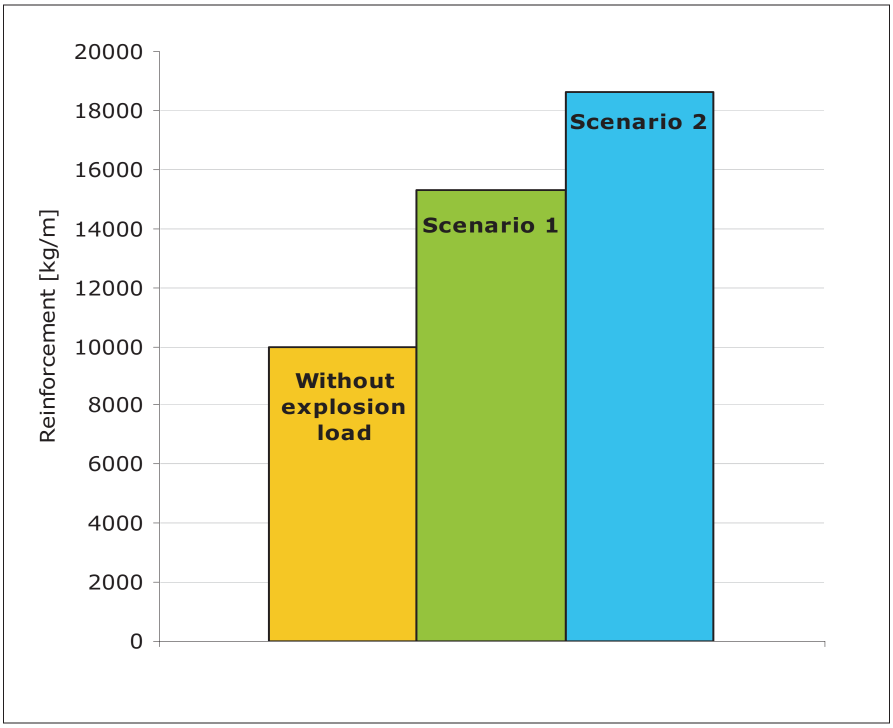


Figure 5-16 Required amount of reinforcement for different scenarios

It can be concluded that the amount of reinforcement increases to a large extent if the explosion loads have to be withstood. Scenario 1 results in over 50 % additional reinforcement whereas scenario 2 requires over 80 % extra reinforcement compared to the situation in which no explosion load is taken into account.

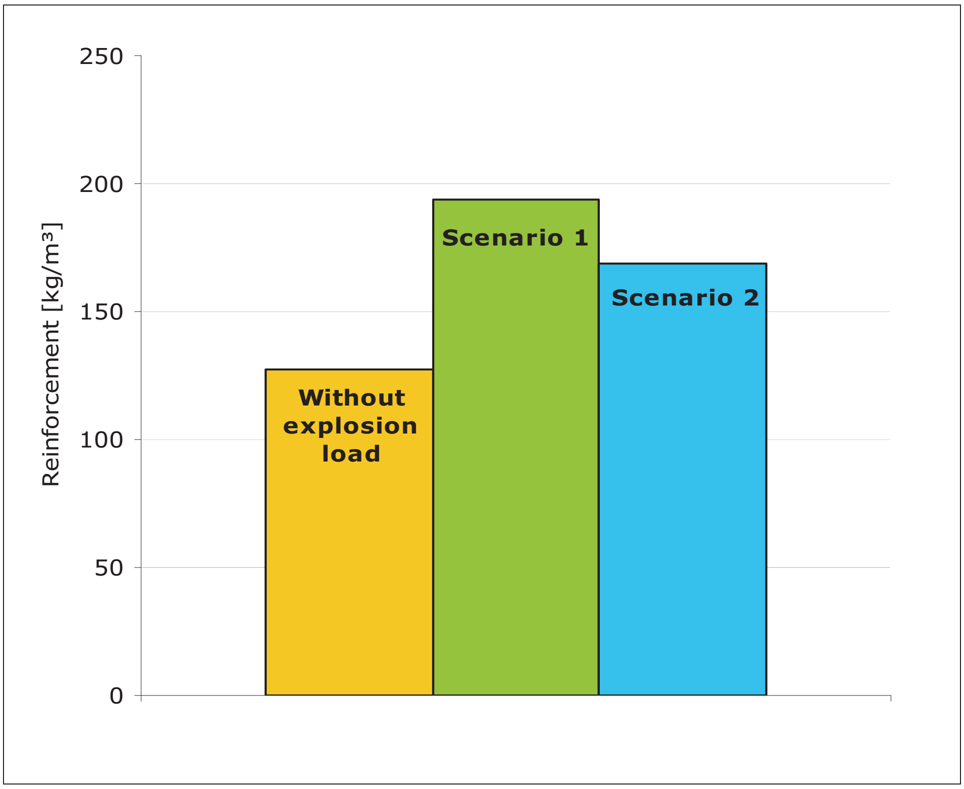


Figure 5-17 Ratio reinforcement – concrete for the different scenarios

The ratio reinforcement – concrete increases also considerable. Since the initial cross-section is just of sufficient dimensions for scenario 1, a very high ratio is found here. For scenario 2, the cross-section is adapted. The thicknesses are increased slightly, therefore a lower ratio is found here while more reinforcement is applied.

The costs involved with a tunnel project are composed off several components. The contribution of reinforced concrete to the total costs of a tunnel is in the order of 30 % [14]. The costs for reinforced concrete fall apart in three components as indicated in the figure below.

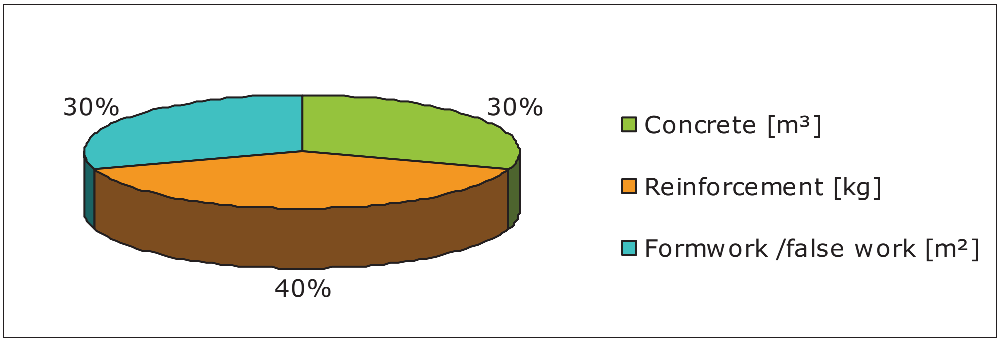


Figure 5-18 Cost components reinforced concrete

Apart from the increased amount of material, a significant larger amount of reinforcement per cubic meter should be applied. Therefore the constructability of the elements will be also less efficient, which will result in a longer construction time and increased costs.

5.6 *Conclusions*

From the foregoing paragraphs a number of conclusions are drawn, these are listed below.

- An explosion load of 500 kPa outward and 300 kPa inward directed is dominant compared to all remaining loads for a representative immersed tunnel.
- Due to the high explosion load, the strength of the element becomes normative, whereas this is hardly ever the case for an immersed tunnel.
- Due to the contrary requirements that hold for the transport and final phase, it is hard to find a suitable solution.
- If the requirement concerning explosion of 500 kPa outward directed and 300 kPa inward directed has to be met, the cross-section should be adapted considerably to comply with the requirements in the several stages. This results in a less efficient solution.
- For an explosion load of 300 kPa outward directed and 180 kPa inward directed, a cross-section of regular dimensions, provides just sufficient capacity.
- The effect of the explosion load on the total costs for the project is considerable.
- Since the considered requirement is only of small magnitude, compared to the representative BLEVE load according to TNO, it will be hard to comply with this higher load.

6 Alternatives

In this chapter possible solutions for designing an explosion resistant immersed tunnel will be explored and discussed. The purpose is to make a selection of promising alternatives that could be investigated further.

6.1 *General*

From the foregoing chapter it became clear that providing sufficient capacity by using the traditional design methods results in inefficient solutions, which is schematically presented in figure 6-1.

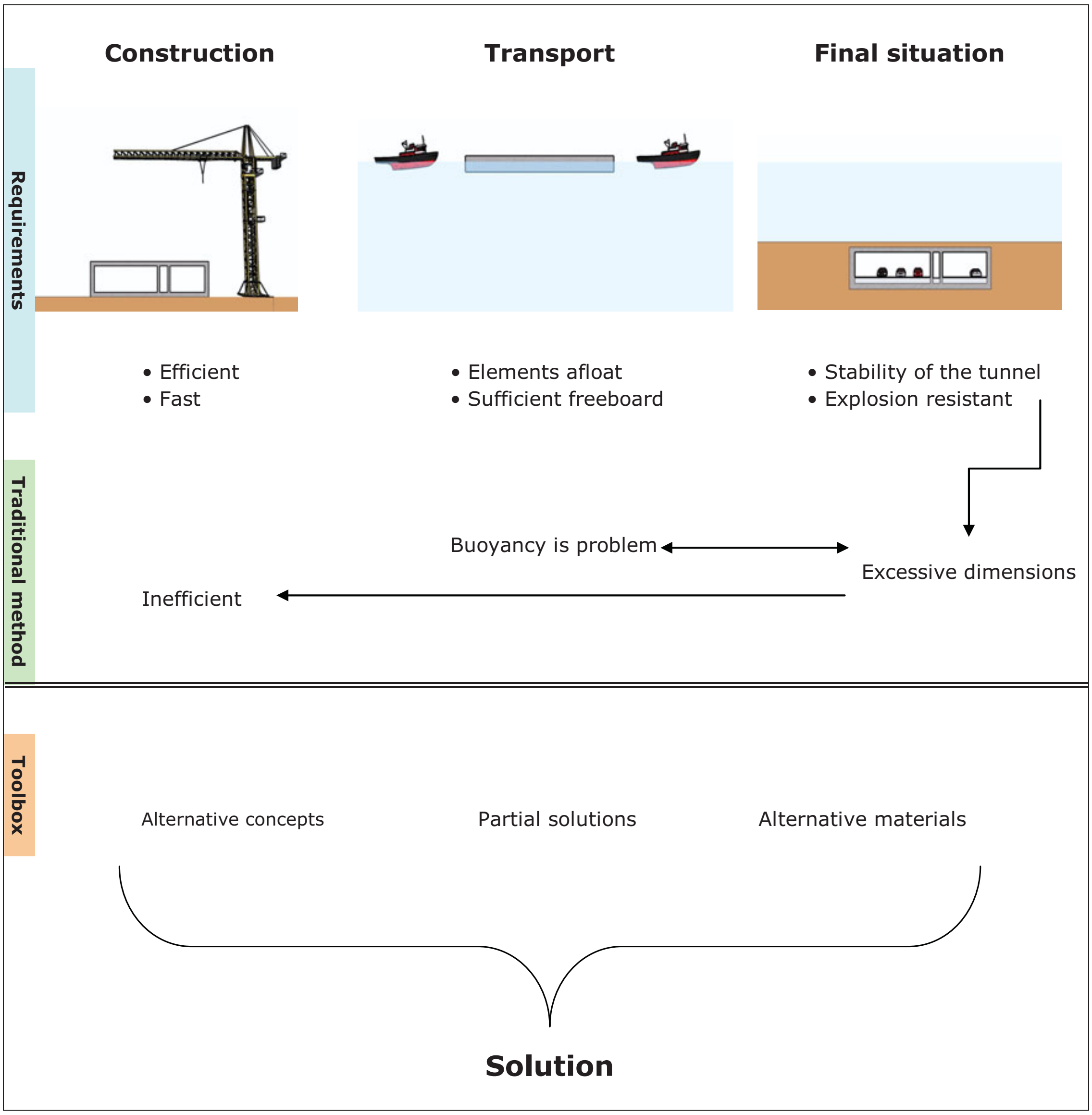


Figure 6-1 Schematically presentation of the design problem

In order to find an efficient solution for an explosion resistant immersed tunnel, several alternatives are developed which are the tools to solve the problem. These alternatives will be discussed in the following paragraphs. The purpose of this part of the research is to explore innovative solutions.

6.2 *Framework*

The development of possible solutions should be done within certain boundaries. A few considerations and common lines of reasoning will be discussed in the following paragraphs. As a result a number of starting points to which the alternatives should comply are stated.

6.2.1 Explosion load

From analysis, it became clear that the recently stated requirement concerning explosion loads does not comply at all with the representative BLEVE load according to TNO. The further research will be focused on providing a solution for the load case as stated by TNO, described in paragraph 4.5.1.

During the development of solutions, it is however kept in mind that the requirement is stated for the Oosterweel tunnel is a relevant problem for BAM Infra. Since the order of magnitude for this load is considerably less, some solutions may be applicable to comply with this requirement. This will be indicated.

6.2.2 Vehicles

One method to limit the load that may occur is to make the requirements for the design of trucks that transport hazardous goods stricter in order to limit probability as well as the magnitude of a potential explosion. Adaptations in the design of the trucks, like the application of pressure relieve valves and special coatings could provide a significant reduction of the risk. Since the number of vehicles is only limited, this could be a beneficial solution. Besides this, existing tunnels that are not designed for high explosion load could be used by the adapted trucks as well. Within the framework of this research this principle is however not considered.

6.2.3 Different concepts for immersed tunnels

In the Netherlands, an immersed tunnel is generally composed of box shaped reinforced concrete elements. Different cross-sections for immersed tunnels are applied worldwide as indicated in the figure below. For normal conditions these deviant cross-sections are not preferable for the Dutch circumstances. This might be different if an explosion load has to be taken into account.

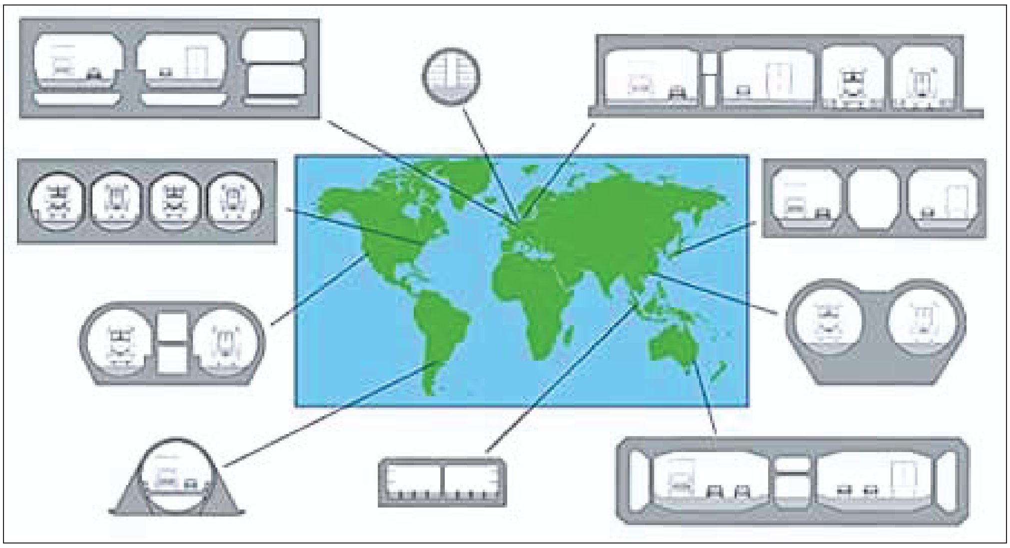


Figure 6-2 Different cross-sections for immersed tunnels worldwide [25]

Different circumstances, strongly dependent of the location result to this variety of solutions. Safety policy is also of influence on the cross-section. In the Netherlands usually an emergency corridor is present between the tubes for example, whereas this is not often accepted in foreign countries. Within the frame work of this research, a qualitative analysis should be made at least.

Circular steel tubes

It should be noticed that in some parts of the world circular tubes are applied instead of rectangular shaped ones. A circular shaped tube seems to comply better with the unidirectional pressure. In case of an explosion there will be a large tensile force present in the lining instead of bending moments, the following relation holds.

$$\sigma_r = \frac{P \cdot D}{2 \cdot t} \cdot 10^3 \qquad (6.1)$$

Where:

σ_r	=	The radial stress in the lining	[N/mm^2]
P	=	The occuring pressure	[kPa]
D	=	The diameter of the tube	[m]
t	=	The thickness of the lining	[m]

If multiple lanes are required, a circular tube results in a very large diameter. The hollow space is not used efficiently in this case as illustrated in figure 6-3, in which the space that is not used is printed grey.

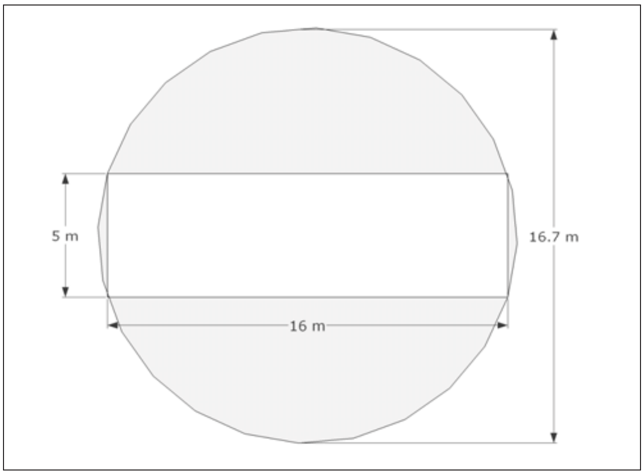


Figure 6-3 Circular cross-section

Due to the large height of a circular cross-section, the level at which the tunnel should be constructed must be lower, leading to additional dredging and longer entrance structures. In the Netherlands the crossings are relatively short which makes this effect even less favourable. Alternatively separate circular tubes can be made in order to limit the height of the element. The accommodation of emergency corridors is in this case difficult or inefficient however. Concerning the resistance against explosions, a large cross-section is favourable, since it reduces the peak pressure that can occur. Though, these effects are considered to be only limited and outweighed by the mentioned drawbacks for large cross-sections.

Steel box tunnel
There is also some preliminary research performed to the construction of a steel tunnel. There are mainly benefits for this alternative if the waterway that has to be crossed is shallow, since the elements have a small draught compared to those for which reinforced concrete is used. The water tightness of the tunnel is strongly influenced by the quality of the welding and may be a problem. Because of the increasing price of steel this solution less is economical. Furthermore, the preservation of the structure is an important issue.

Steel-concrete-steel box tunnel
This concept is studied recently in the Netherlands. The cross-section of this alternative is composed of sandwich panels, consisting of two steel plates with concrete in between. There are several advantages connected to this alternative, like the quick manufacturing of the elements and the fact that no formwork is required. In Japan, this principle was used in a pilot project, though it appeared to be very expensive compared to the conventional method. Especially the steel and welding increased the costs to a great extent. Furthermore, the structure should be preserved by means of cathodical protection for example. Besides this, fire resistant coating is relatively difficult

to apply on a steel surface. It is decided not to consider this principle any further within the framework of this research.

Conclusion
There are no obvious advantages for one of the considered concepts. Furthermore, it is preferred by BAM Infraconsult to consider the usual Dutch solution for the cross-section of an immersed tunnel as a starting point for the research, since there is a lot of experience gathered within this method and processes are standardised to a great extent.

6.2.4 Acceptability of local failure

From the paragraph 4.5 it can be concluded that the order of magnitude of the load in the vicinity of the BLEVE is of severe magnitude. It seems to be hard to provide sufficient structural capacity to withstand this load at first glance. Since the extreme peak applies rather locally, it could be considered to design the tunnel for the load that occurs 20 meter from the centre of the BLEVE and accept severe damage or even failure locally, in the vicinity of the BLEVE. The idea may rise to make the design in such a way that in the event of an explosion the tunnel will be damaged, though repairable. In this way the out of service time after an explosion may be shortened significantly, compared to the situation in which the complete tunnel will be destroyed. This is a rather radical assumption to which a number of important consequences are connected. These will be discussed briefly now.

Due to the excessive magnitude of the peak load, failure is to be expected locally. This will result in inundation of the tunnel. As a result of effects of the explosion and inundation, the complete interior of the tunnel as well as the installations are expected to be destroyed. Besides this, the water that enters the tube results in a significantly increased foundation pressure. This may cause large settlements, endangering the overall stability.

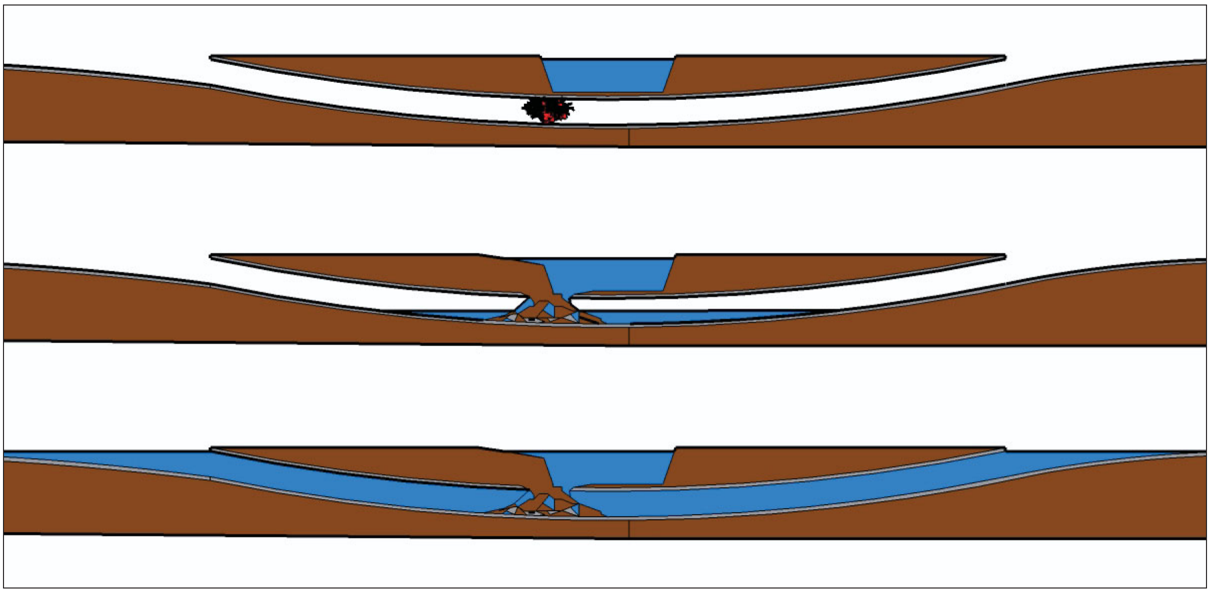


Figure 6-4 Inundation of the tunnel in the event of an explosion

In order to make repair possible, there should be some remaining structural capacity. In longitudinal direction, a compressive force is present in an immersed tunnel. This force originates from the immersion process, whereby the elements are pushed to one and other by the hydraulic pressure. The presence of this axial force ensures an appropriate linking between the elements. In case part of the tunnel will be completely destroyed the stability is no longer ensured and total failure of the structure will occur.

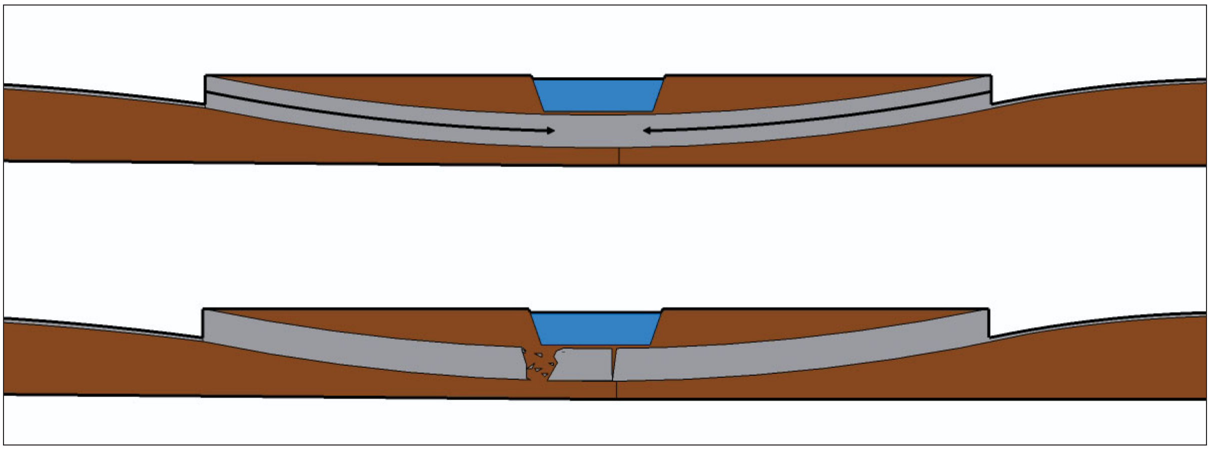


Figure 6-5 Instability as a result of lost immersion force

In order to transfer the immersion force, it is required that at least a part of the cross-section remains intact. The floor slab and outer walls are important in this respect. In order to make the structure repairable, it can be designed to fail in a desired manner in the event of an explosion. Certain elements can be designed with significant lower structural capacity on purpose, in order to make those fail first and make sure that the crucial parts are relieved and remain intact.
After the explosion, the gab should be isolated and the tunnel should be pumped dry. Subsequently, all rubble and wrecks should be removed from the tubes. The structure can be repaired. Finally all installations should be replaced. Furthermore, the tiles, fire resistant coating and asphalt should be applied. These activities will require considerable effort, though the out of service time of the connection will be significantly less compared to the situation in which the complete tunnel has to be replaced. The operations are complicated however and it is not desirable to make these concessions to the structural performance of the tunnel in this early stage of this research, therefore this principle will not be considered further.

6.2.5 Starting points

From foregoing considerations a number of conclusions are drawn. These are starting points for the continuation of the research together with a few other relevant aspects that are determined. These starting points are listed below.

- The alternatives will focus on the design of the tunnel, adaptations of vehicles are not considered although this may provide an efficient solution.

- In consultation with BAM Infraconsult it is decided to focus on a box shaped concrete cross-section for the immersed tunnel, since this is the commonly applied and therefore most relevant concept for the Dutch circumstances.

- An immersed tunnel can be designed in such a way that locally failure occurs, though repairing the structure is possible. This is a rather radical approach however with striking consequences which is not preferred.

- It is decided that the plastically bending moments should not be exceeded as a result of the explosion load.

- The study will primarily focus on the immersed part of the tunnel, since it is presumed that finding a solution for the entrances is less complex.

- In case an explosion of determined magnitude occurs, the structure should not collapse. Besides this, it should be reasonably repairable.

- Since an immersed tunnel is composed of several segments which are exposed to different loads as a result of the variety in depth and location, provisions may differ per segment. In this manner a more optimal solution in an economical sense might be achieved.

- It is desirable to remain the traditional solution as far as possible in order to build efficiently and take advantage of the available experience.

6.3 *Development of the alternatives*

In order to find innovative solutions, it is attempted to consider the problem widely. There are several ways to withstand the severe explosion load. Obviously a combination is also possible. The alternatives that will be considered in the following paragraphs are listed below.

1. Reduction of the load
1.1 Increasing the cross-section
1.2a Weak areas in the roof
1.2b Weak areas in the intermediate walls

2. Dissipation of energy
2.1 Application of secondary walls
2.2 Absorbing coating
2.3 Sandwich structure

3. Increasing the capacity
3.1 Structural floor in the emergency corridor
3.2 Application of pre stressing
3.3 Application of fibre reinforced polymer rebar
3.4 Application of external reinforcement
3.5 Application of fibre reinforced concrete

4. Separate tubes

5. Weight limiting measures
5.1 Low density concrete
5.2 Hollow spaces
5.3 Staged construction

These alternatives will be described qualitatively and the advantages and disadvantages will be discussed. The most promising and or interesting solutions will be selected for further investigation.

6.4 *Reduction of the load*

The structure itself is also of influence on the load, especially the cross-section is determining for the occurring pressures. With this in mind, a number of alternatives is developed.

6.4.1 Alternative 1.1: Increasing the cross-section

There exists a relation between the occurring pressure and release time. Furthermore, the cross-sectional area of the tunnel tube is of importance since reflection results in high pressures. Increasing the cross-section may lead to a more favourable situation. In figure 6-6 the relation between the occurring pressure as a function of the release time for a 50 m³ LPG BLEVE is depicted for an open half space and a tube with a cross-section of 60 m². This example is copied from [12].

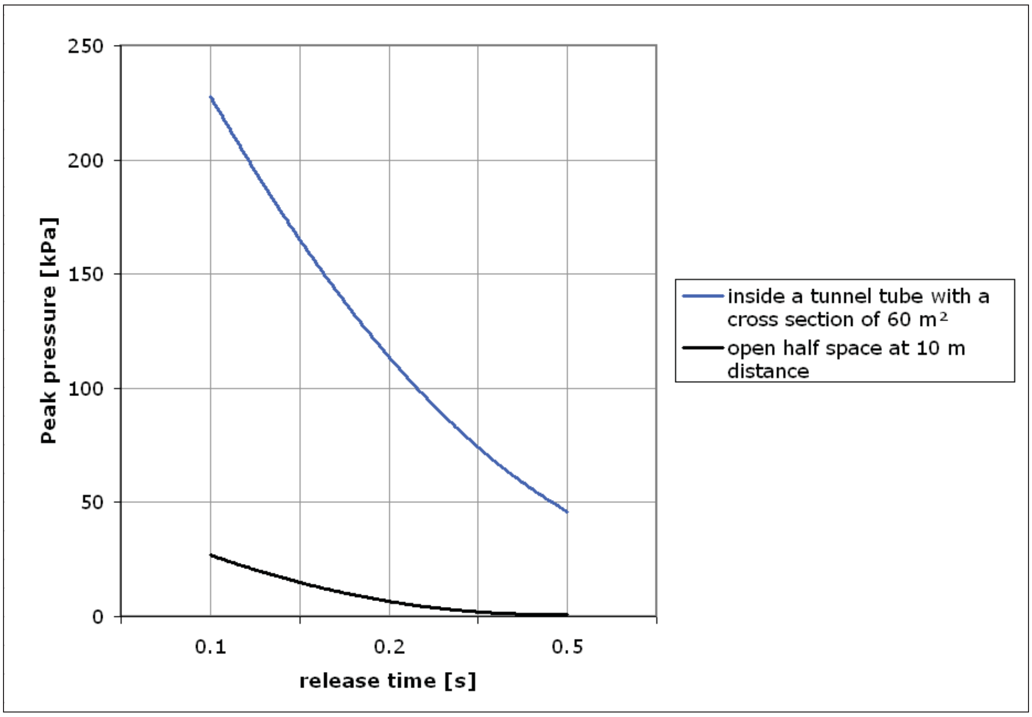


Figure 6-6 Influence of the cross-section on the peak pressure

It can be concluded that a larger cross-section significantly reduces the load, especially if the release time is short.

6.4.1.1 First evaluation

Since increasing the height strongly affects the length of the tunnel, this should be avoided as far as possible. Increasing the width seems a possibility, the bending moments for the roof and floor slab will be larger as a result however. The pressure is presumed to decrease proportional with an increasing cross-section for the considered range, while the bending moments increase proportional to the square of the span. In order to show the influence of increasing the width, a 15 m wide tunnel with a height of 5 m is used as starting point. The height remained constant whereas the width is increased in steps of 1 m, the results are listed in the table below.

w [m]	h [m]	cross-section [m²]	Δq [%]	ΔMroof [%]	ΔMwall [%]
15	5	75	0	0	0
16	5	80	-6	7	-6
17	5	85	-12	13	-12
18	5	90	-17	20	-17
19	5	95	-21	27	-21
20	5	100	-25	33	-25

Table 6-1 Influence increased width on bending moments

Increasing the cross-section by enlarging the width, results in a decreased bending moment for the walls, whereas the bending moment for the roof will be larger. It can be concluded that this is not an efficient way to reduce the load. Besides this, increasing the cross-section requires additional materials and activities.

Alternatively, the height of the tunnel can be increased in order to enlarge the cross-section. Similar evaluation for this option is made, the results are listed in the table below.

w [m]	h [m]	cross-section [m²]	Δq [%]	ΔMroof [%]	ΔMwall [%]
15	5	75	0	0	0
15	6	90	-17	-17	20
15	7	105	-29	-29	40
15	8	120	-38	-38	60
15	9	135	-44	-44	80
15	10	150	-50	-50	100

Table 6-2 Influence increased height on bending moments

Increasing the height leads to reduction for the roof slab, whereas the situation for the intermediate wall becomes less favourable. Furthermore, the length of the tunnel will increase as a result of the greater depth. This is because of limitations considering the maximum curvature of the longitudinal alignment of the tunnel. These limitations ensure suitable lines of sight and a comfort for the users of the tunnel. With maximum inclinations, a deeper tunnel will obviously result in a longer connection. This results in additional use of material and a longer construction time. Apart from that, the amount of material that should be dredged increases.

6.4.1.2 Conclusion

Increasing the cross-section of the tunnel in order to reduce the load is considered to provide no efficient solution. The alternative will not be further considered within the framework of this research. The following aspects are important in this respect.

- Although the load will reduce for a larger cross-section, the spans increase. This result in bending moments of a larger magnitude.
- The required amount of materials will increase to a large extent

6.4.2 Alternative 1.2A: Weak areas in the roof

Reduction of the load could be achieved by providing the possibility to let the pressure escape partly. In high speed rail tunnels, a pressure relieve system is sometimes applied. Shafts are made on the roof, connecting the tube with open air through which the pressure can escape. Since often important navigation routes have to be crossed, it is not possible to apply shafts there. It may be beneficial for the entrances though.

In the figure below, a typical longitudinal cross-section is depicted in which the immersed part and the part is constructed by means of cut and cover are indicated.

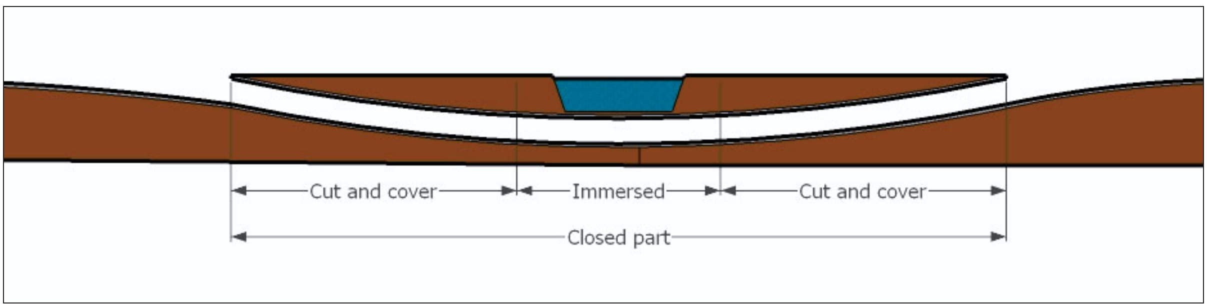


Figure 6-7 Different sections of an immersed tunnel

6.4.2.1 First evaluation

The closed part of a tunnel often extends beyond the banks of a waterway. The reason for this is in many cases of esthetical and structural nature. These cut and cover parts of the tunnel are most vulnerable to explosion loads, because they are situated shallower and thus are subjected to relatively small surcharge loads. If these parts of the tunnel could be built without a roof, the pressure as a result of an explosion does escape easily. Alternatively, a light roof structure can be applied which fails instantaneously in case of an explosion. In this way, the problem will reduce to the part of the tunnel that is immersed. Large retaining walls are however required in that case, as indicated in figure 6-8.

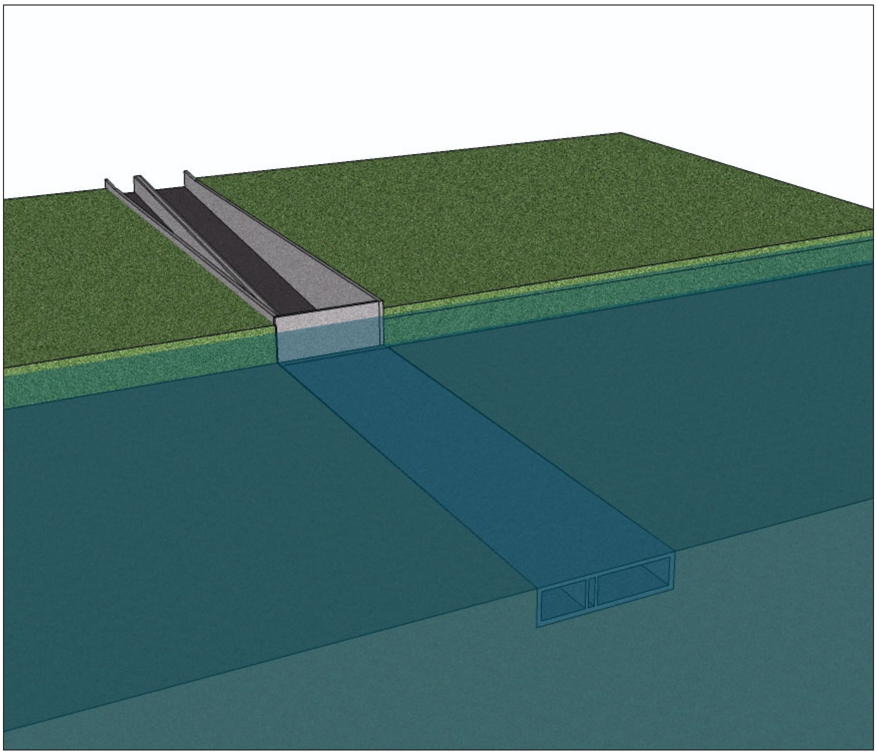


Figure 6-8 Open entrance

In case the entrance is open, there is no surcharge load consisting of soil. This is unfavourable for the vertical stability, the element tends to float up. Therefore the tension piles that support the entrance should have a larger bearing capacity or additional piles have to be applied.

The transmission of the immersion force that is present in the longitudinal direction of the tunnel requires special attention. The immersed elements are pushed against the other by the water pressure, which introduces axial forces in the longitudinal direction of the tunnel as indicated in figure 6-9.

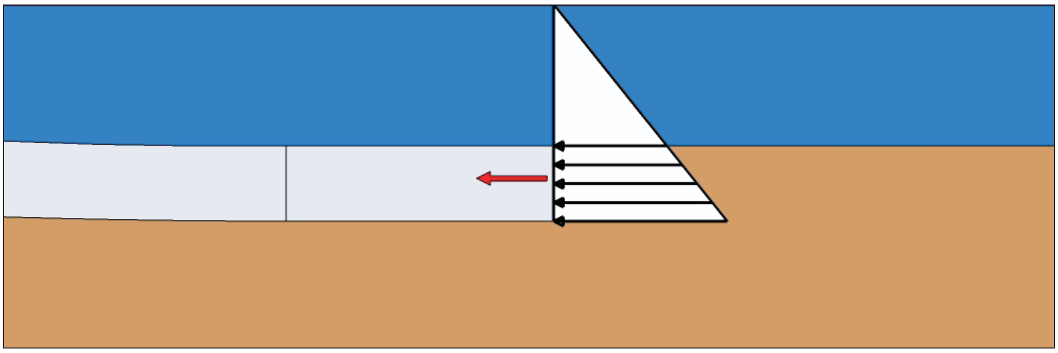


Figure 6-9 Immersion force

The immersion force will be always present in the tunnel. It should be transferred via the elements to the abutment. Usually, the cut and cover section is supported by piles. The immersion force is transferred to the subsoil via the substructure. The transition between the immersed part and the cut and cover section should be designed in such a way that the forces are introduced without causing large bending moments locally.

For the part of the tunnel that is located under the waterway, it might be possible to create weak areas that fail on purpose if an explosion occurs. Weak areas can be of structural nature, a reduced amount of reinforcement could be applied locally for example. Furthermore, the thickness of the structure may be smaller. These weak areas can be made in the roof for instance, as indicated in the figure below.

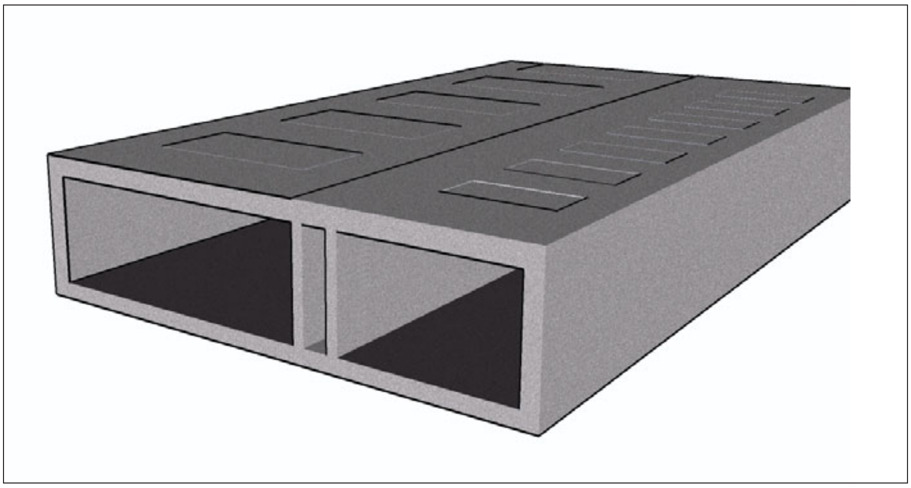


Figure 6-10 Weak areas in the roof slab

In case of an explosion, locally, controlled failure will occur whereas the pressure will be released. The weak areas should be the first locations where damage occurs. Therefore, the remaining parts of the structure should have considerably more capacity to prevent total failure. Besides this, it should be determined at which pressure these spots should fail, or in other words what load has to be withstood by the rest of the structure. It should be noted that the weak areas should have at least enough capacity to withstand the surcharge load. Since at the deepest parts of an immersed tunnel usually a large surcharge load consisting of water and soil is present, it is questionable to what extent an effective release of pressure can be achieved in this way. Failure of the weak areas should be very fast, since the duration of the load is also very short. Since the weak areas are only of limited size, reflection of the pressure may still occur, resulting in relatively high loads. Extensive research is required to determine the effect that can be achieved with this method, furthermore the effectiveness will probably be dependent on the local circumstances, like the water depth to a great extent. Another important drawback is the fact that the tunnel will inundate as a result of an explosion. Afterwards, the gaps can be repaired with help of divers and the tunnel can be pumped dry, though considerable damage is to be expected. Furthermore the tunnel will be out of service for months at least.

6.4.2.2 Conclusions

A few important conclusions can be drawn from foregoing considerations, these are listed below and distinction is made between the cut and cover section and the immersed section of the tunnel.

Cut and cover section

- It could be beneficial to build the cut and cover section of the tunnel without a roof or apply a low strength roof here. In case of an explosion the pressure will be released easily and the design problem is reduced to the immersed part.
- Transmission of the immersion force through the cut and cover section can be accommodated by means of a grid of beams in case a roof is absent.
- A drawback is that the retaining walls should be of considerable height.

Immersed section

- Since there is a surcharge load of considerable magnitude present the failure probably occurs not fast enough to release the structure.
- Extensive research to the effect of this principle is required. Probably the effects are dependent on local circumstances like water depth. These may even vary strongly within one project.
- After an explosion the tunnel will inundate and repairing the structure leads to an out of service time in the order of months at least.

6.4.3 Alternative 1.2B: Weak areas in the intermediate walls

An option is to make weak areas in the intermediate walls. In case of an explosion one or both walls will be blown out, increasing the cross-section and in this way a reduction of the occurring pressure will be achieved. Besides this, during the destruction of the weak areas energy of the explosion will be dissipated. Since the intermediate walls support the roof, these should be designed in such a way that in case of an explosion this function can still be fulfilled. In order to achieve this, it is possible to make columns with a beam on top and low strength panels in between. An indication is given in the figure below.

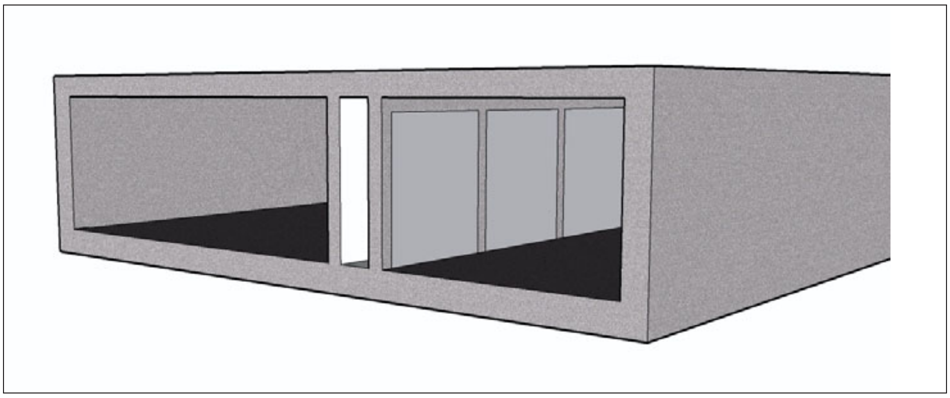


Figure 6-11 Weak areas in the intermediate walls

It is presumed that increasing the cross-section has the most effect on the reduction of the pressure and therefore this alternative is most effective if the design is made in such a way that both intermediate walls will be destroyed. In the figure below the principle of this solution is depicted.

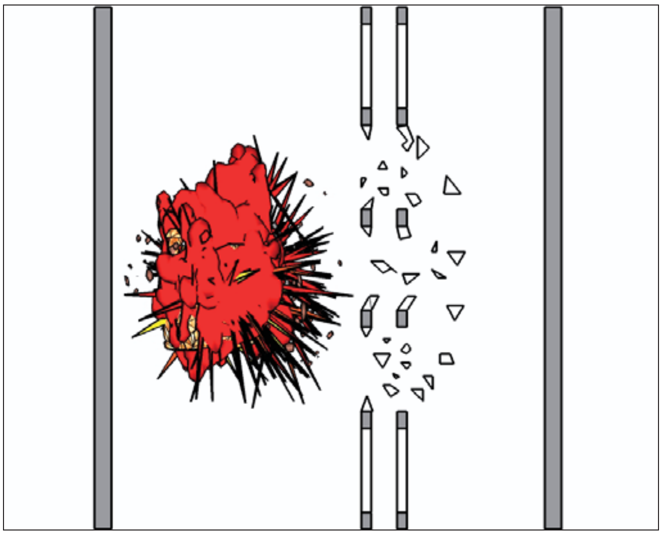


Figure 6-12 Principle of weak areas in the intermediate walls

In correspondence with the analysis made for alternative 1.1, estimation can be made for the reduction of the bending moments that occur. For simplicity it is assumed that the cross-section doubles as a result of the destruction of the walls. This will result in a reduction of approximately 50 % for the bending moments that may occur in the roof. Dissipation of energy as a result of this destruction is not considered. Of coarse this is a very rough estimation, which is probably very optimistic.

6.4.3.1 Conclusions

There are a few important conclusions to be drawn for this alternative, these are listed below.

- Reduction of the load can be achieved in this way since the bending moments that occur also will decrease.
- Since the intermediate walls are not exposed to permanent horizontal loads, fast failure may be easier achieved than for weak areas in the roof slab.
- The low strength panels, barely need to have any structural capacity, these will provide just a boundary between the tubes.
- Usually, cables for electricity and communication run through the emergency channel, these will be destroyed in the event of an explosion.
- The problem will be expanded to the second tube, resulting in more casualties and additional damage.

6.5 *Dissipation of energy*

It could be investigated to dissipate energy partly in order to relieve the structure.

6.5.1 Alternative 2.1: Absorbing coating

The idea may rise to cover the inside of the tube with a material that reduces the load by dissipation of energy.

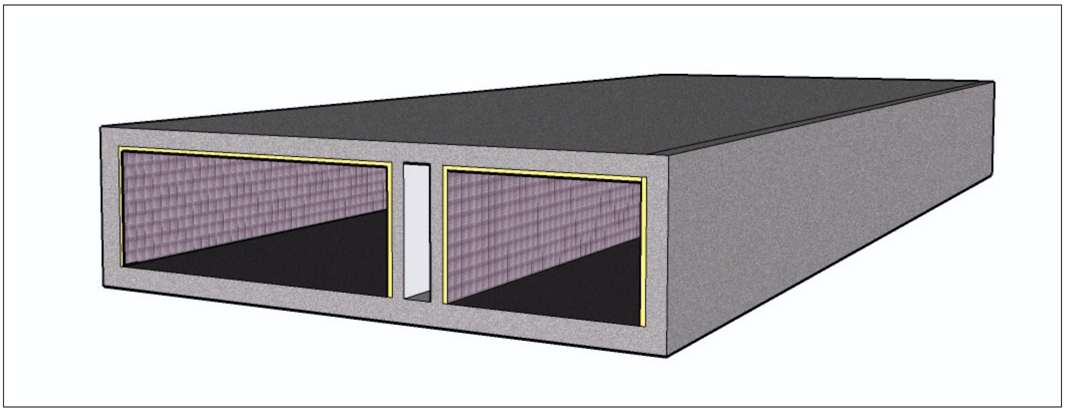


Figure 6-13 Absorbing coating

The luggage in airplanes is often stored in explosion resistant containers nowadays. These containers are apparently made of materials that are capable of absorbing a large amount of energy and preventing an airplane to be dangerously damaged. There seem to be similarities between the storage of luggage in an explosion resistant container and designing an explosion resistant tunnel. The material may absorb the load to a large extent but there will always be a supporting force acting on the structure however. Since the order of magnitude of the occurring pressure due to a BLEVE is severe, providing sufficient structural capacity will still be a problem. Apart from that the spans of the tunnel are very large compared to the dimensions of a blast resistant container, which makes it unrealistic that similar measures will provide a solution.

6.5.2 Alternative 2.2: Secondary walls

Secondary walls inside the tube, which may deform to a large extent or even fail in case of an explosion, could be considered. A disadvantage is that the cross-section needs to be wider in order to provide space for the walls. In the event of an explosion, there may be fragments of the wall that are launched and cause damage. It should be noted that this solution only relieves the walls.

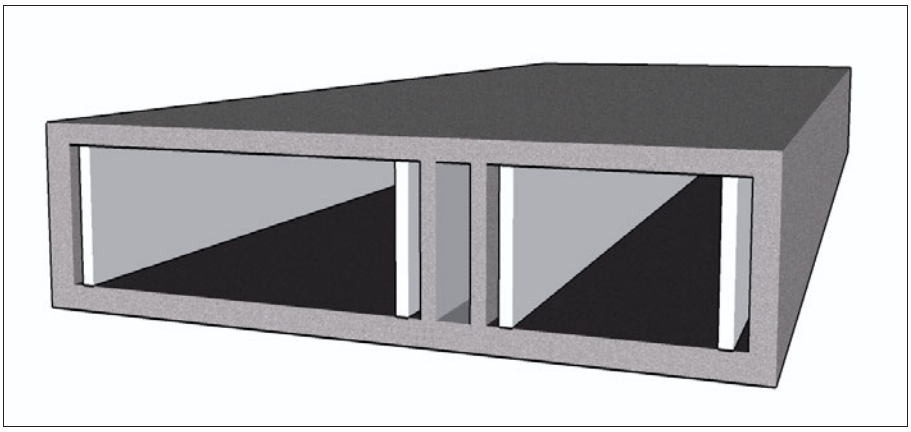


Figure 6-14 Secondary walls

6.5.2.1 Conclusions

The following conclusions can be drawn for this solution.

- The primary walls are not directly exposed to the load.
- Due to dissipation of energy the entire structure will be relieved.
- The width of the structure will be increased.
- The purpose is to isolate the problem in the tube where the explosion occurs.

6.5.3 Alternative 2.3 Sandwich structure

Application of a secondary tube inside the tunnel may provide significant energy dissipation. The space between the two tubes can be filled with a material that provides damping, like sand for instance. The principle of this alternative is presented in figure 6-15. It should be noted that the dimensions of the element will be increased significantly.

In the early eighties, tests were performed with panels that should relieve the primary tube. These tests were based on gas explosions with small magnitude only. Different types of panels and fill material were considered. The effectiveness of the investigated solution appeared to be limited, though it is concluded that building a tunnel inside a tunnel instead of the application of separate panels would be more beneficial. This solution was not studied any further however, though it may be a suitable alternative.

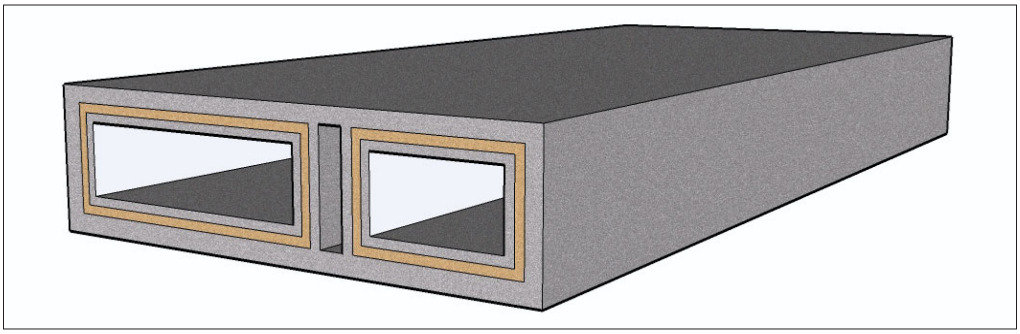


Figure 6-15 Sandwich structure

In the event of an explosion, the inner tube will be damaged or even destroyed, whereas the primary tunnel remains in tact as indicated in the figure below.

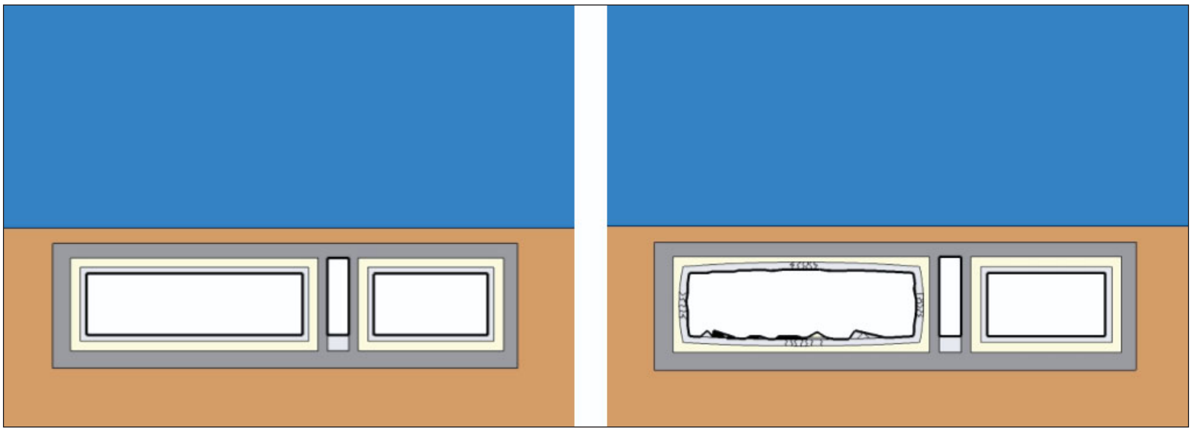


Figure 6-16 Effect of an explosion in the left tube

- The inner tube and fill material provide resistance against the explosion and relieve the actual structure.
- In the event of an explosion, the secondary tube may be destroyed completely, as long as the primary tube remains undamaged.

6.6 *Increasing the capacity*

The resistance of the structure can be increased in many ways. One of the starting points for the development of alternatives is to maintain the traditional cross-section as far as possible. Several possible solutions are considered in this respect.

6.6.1 Alternative 3.1: Structural floor in the emergency corridor

As stated before, the intermediate walls are weak links with respect to resistance against explosion loads. The corridor between these walls usually fulfils two functions. The lower part provides an escape route for people inside the tunnel in case of an emergency. In the higher part, cables for electricity and communication run through this small tube. Usually, a non structural floor is present about halfway and the available space is used efficiently. The resistance of the intermediate walls can be increased to a great extent if the floor is designed to be a structural element that limits the span of each wall as indicated in the figure below. Since this provides only extra resistance for the intermediate walls, it is more a partial solution than an alternative.

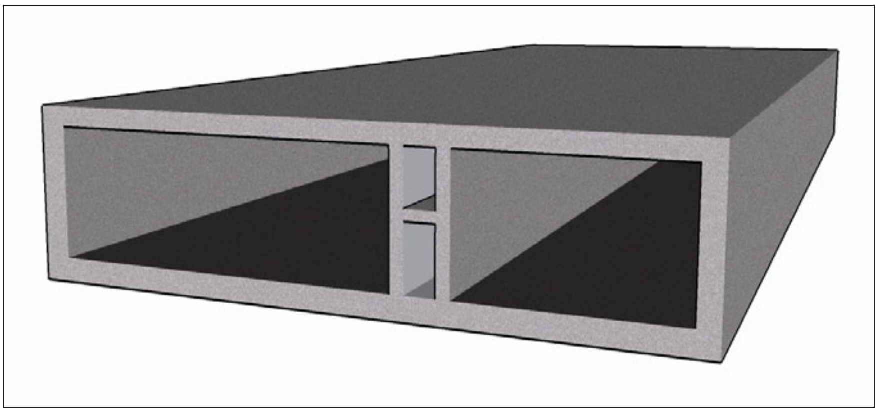


Figure 6-17 Structural floor in emergency corridor

6.6.1.1 First evaluation

In order to investigate the effect of this measure, a simple calculation is made. Initially, intermediate walls with a height of 6 m and without a structural floor in between are considered. A uniformly distributed load of 100 kN/m is applied to one of the walls, as indicated in the figure below. The occurring bending moments are calculated with the software package ESA.

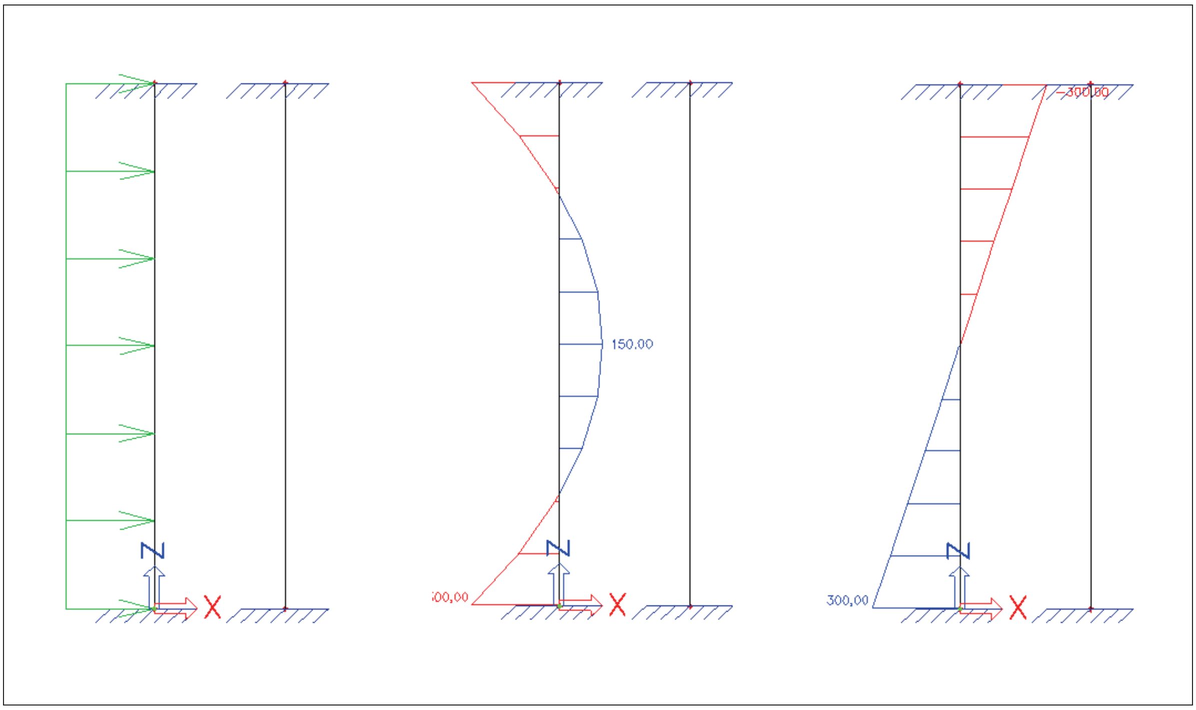


Figure 6-18 Bending moments and shear forces without structural floor

Subsequently a structural floor is applied halfway the walls, as indicated in the figure below.

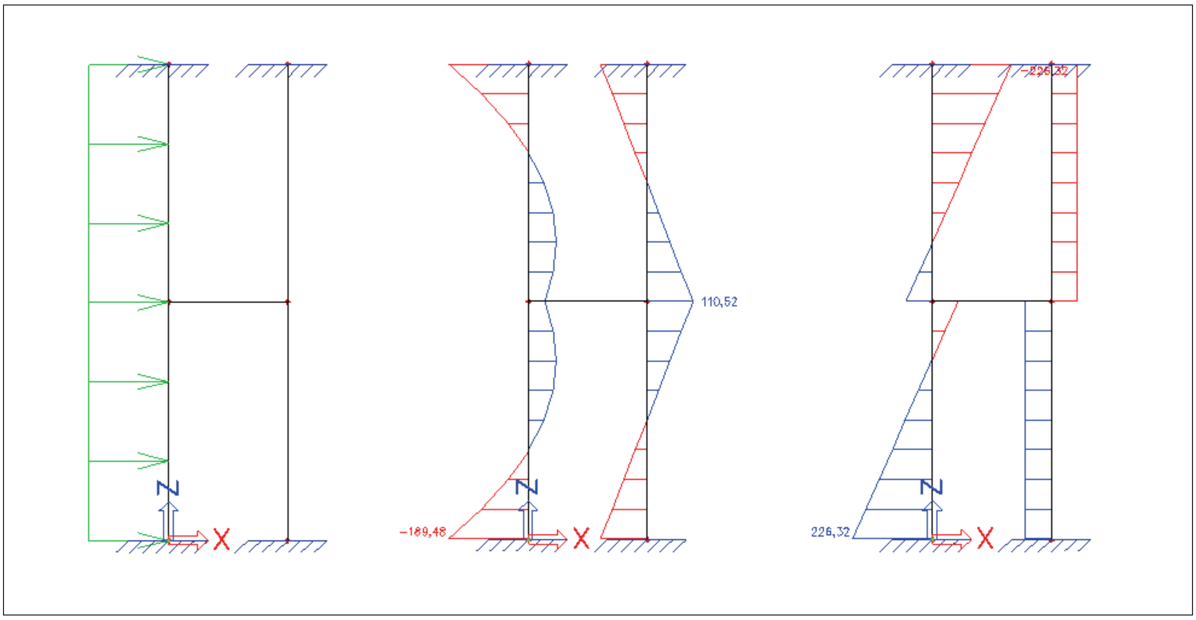


Figure 6-19 Bending moments and shear forces with structural floor

It can be concluded that application of a structural floor results in a reduction of the occurring bending moments with a magnitude of approximately 30%. The shear forces decrease with approximately 25 %. This significant improvement, which can be achieved relatively easy, makes this a promising measure.

6.6.2 Alternative 3.2: (Partly) pre stressing

A method to increase the capacity of the structure is to apply pre stressing. The principle of this alternative is to introduce compression stresses in the concrete in advance. Since concrete has a relatively low tensile strength whereas the compressive strength is rather high, this is an efficient principle. There are a several methods to apply pre stressing for concrete structures. The most common are described below.

Pre stressing by means of pre stressed tendons
If this method is applied, steel or synthetic tendons are installed in the formwork. These tendons are tensed by means of jacks, thereafter the concrete is poured. If the concrete is hardened the element is pre stressed, the force in the tendon is transferred by means of bond stresses. This method is very suitable for prefabricated elements.

Pre stressing by means of post stressed tendons
The tendons are placed in synthetic casings which will be installed in the formwork. After pouring and hardening of the concrete, the tendons are tensed. The tendons are attached to special anchor plates which transfer the force to the concrete. The casings can be injected with a special mortar in order to create bond between the tendon and concrete. Another possibility is to fill the casings with grease and completely transfer the stress by means of the anchor plates.

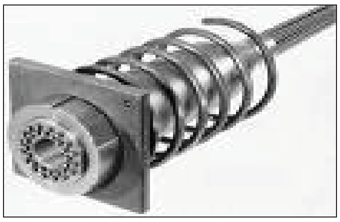


Figure 6-20 example of anchor plate

6.6.2.1 First evaluation

Since the intermediate walls and roof are the weak elements if it comes to resistance against explosion loads, it could be considered to apply pre stressing here. It is likely that increased thicknesses are required as well. The principle solutions for both elements are given in the following figures. Obviously, for the outer wall and floor slab similar measures can be taken, if necessary.

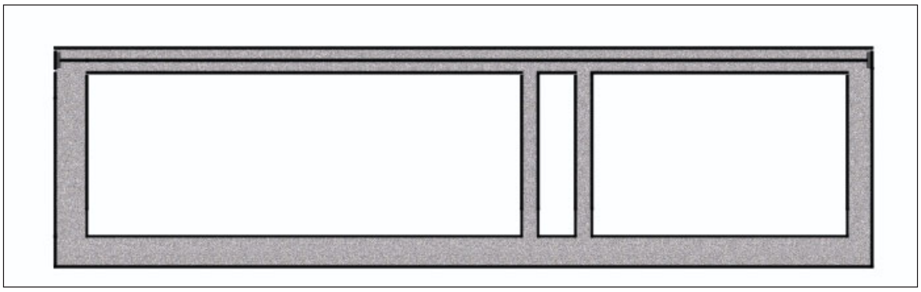


Figure 6-21 Pre stressing of the roof slab

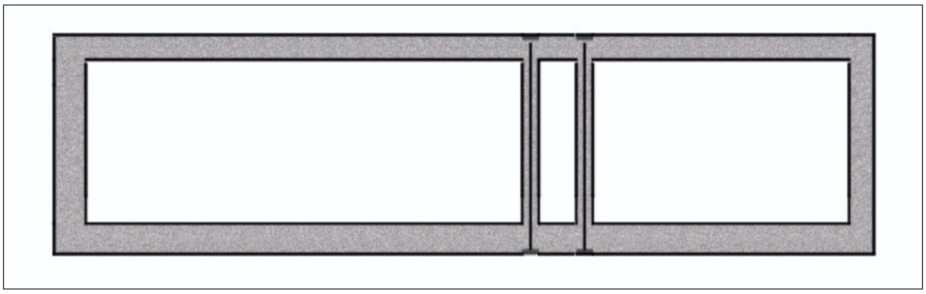


Figure 6-22 Pre stressing of the intermediate walls

6.6.2.2 Conclusions

- The structural capacity can be increased by applying pre stress.
- The effect of this solution is limited by the compressive strength of the concrete.
- For extreme loads it is inevitable to increase the cross-section of the structural elements or the quality of the concrete.
- Pre stressed concreted has a broad spectrum of applications and a lot of experience is gathered within this technique.
- The construction phase will be more complicated than for reinforced concrete.

6.6.3 Alternative 3.3: Fibre reinforced polymer rebar

Traditionally, steel is used as reinforcement for concrete structures. For some applications reinforcement with different specifications is required however. Fibre reinforced polymer rebar is a substitute that is applied on small scale and for very specific situations nowadays. The materials and techniques improve continuously however and there are several manufacturers. In figure 6-23 examples of glass fibre reinforced polymer rebar are depicted.

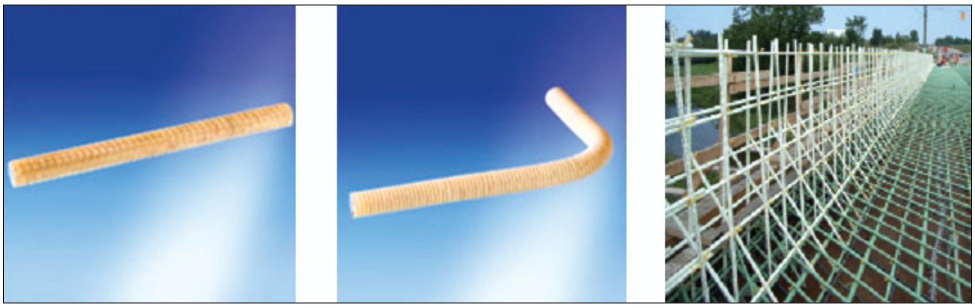


Figure 6-23 Examples of GFRP, ComBAR® [27]

Production

The rebar consists of bundles of fibers, enveloped with epoxy resin. These bundles are given some roughness by means of sand grains or applying some profile. The bars are produced in a factory by means of pultrusion, this process is schematically depicted in figure 6-24.

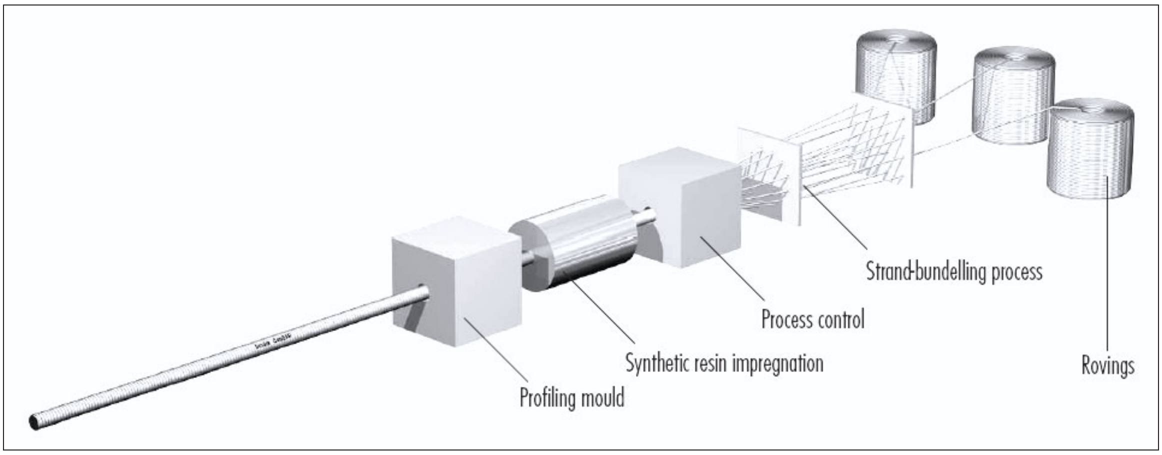


Figure 6-24 Principle of pultrusion [27]

Plastically bending of the bars after production is not possible. For the manufacturing of bended bars, a special die will be used. This process results however in a decreased density of the fibers and thus lowers the strength, which is an important disadvantage.

Diameters varying between 6 mm and 32 mm are available. The materials have advantages for instance if electric transmission through the reinforcement is undesired, steel would interfere signals, structures for which temperature exchange is undesired and structures for which boring through the concrete should be easy.

Some important characteristics of these materials are listed in table 6-3. The values may slightly differ per manufacturer.

Property	AFRP	CFRP	GFRP	FeB 500	Concrete
Tensile strength [N/mm²]	900	2000	650	500	2-5
Compressive strength [N/mm²]	250	250	550	500	25-60
Modulus of elasticity [N/mm²]	50000	120000	30000	210000	25000-36000
Failure strain [%]	2.2	0.017	2.3	0.325	0.01
Density [kg/m³]	1300	1600	1700	7850	2400
Expansion coefficient [10^{-6}/ °c]	-1	0.3	10	12	7-12

Table 6-3 Characteristics of fibre reinforced polymer rebar

AFRP = Aramide Fibre Reinforced Polymer
CFRP = Carbon Fibre Reinforced Polymer
GFRP = Glass Fibre Reinforced Polymer

It should be noted that the application of glass fibre reinforcement is the most plausible since the expansion coefficient corresponds best with concrete.

A disadvantage of these fibre reinforced polymer rebar is that the materials are rather brittle. This is usually a problem because if a structure fails in a brittle way, there will be no large deformation pattern that alerts the people to flee away before collapse. In this case however, the reinforced polymers should be used only as extra for the incidental explosion load. For this type of load it makes no sense to design the structure in such a way that it gives a warning before failure, therefore it is justified to make use of these materials.

Steel rebar has a relatively high deadweight and it is therefore not easy to handle during construction, especially if large amounts are required. Therefore, partly substitution of the steel by relatively light glass fibre or synthetic reinforcement might be advantageous. The tensile strength of these materials is rather high, which makes this a promising alternative at first glance.

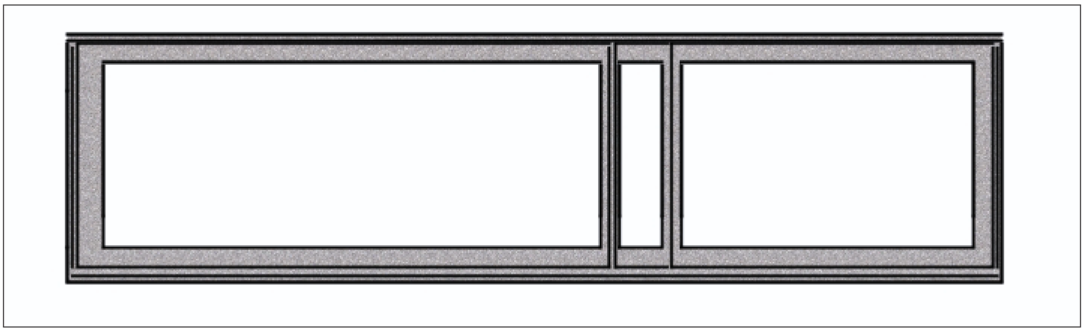


Figure 6-25 Application of fibre reinforced polymer rebar

6.6.3.1 Conclusions

The following conclusions can be drawn with respect to the application of fibre reinforced polymer rebar.

- The materials have a high tensile strength compared to reinforcement steel, therefore less reinforcement should suffice.
- The materials have a small deadweight compared to reinforcement steel, which is advantageous for the construction phase.

- It should be noted that without enlarging the cross-section, the structural capacity can not be increased. Application is therefore possible if the capacity could be also provided with regular rebar.

- There is not much experience gathered with the application of these materials.

- In case of fire, the strength of the materials decreases much faster compared to reinforcement steel.

6.6.4 Alternative 3.4: External reinforcement

Generally, the reinforcement of a concrete structure is cast in. It is however also possible to apply external reinforcement. The principle of this solution for the outward directed load is indicated in the figure below. At the top side of the roof and on the intermediate walls inside the emergency corridor, the reinforcement can be applied.

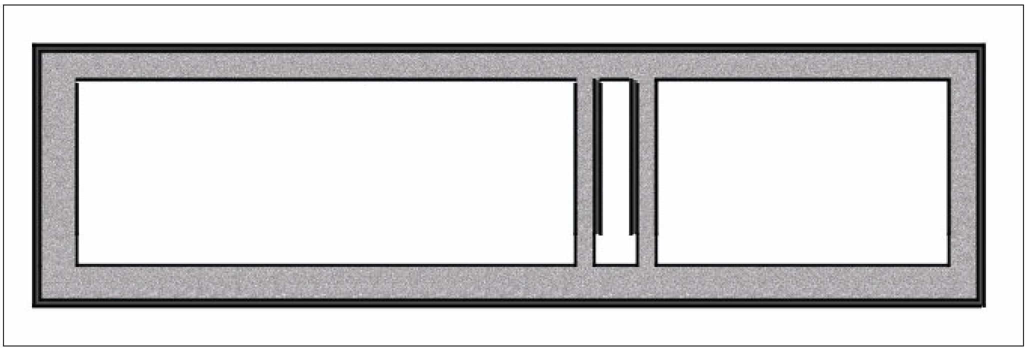


Figure 6-26 Application of external reinforcement

There are two materials that especially have favourable characteristics to be used as external reinforcement, these will be discussed briefly in the following.

Steel plates

- An advantage is that the plates can be used as lost formwork. Connection between the plates and the concrete can be realised by means of dowels.

- Steel plates are not easy to handle, because of their relative high deadweight.

- Steel is susceptible to corrosion and therefore an extensive treatment is required in order to ensure durability.

Carbon fibre lamellas

Carbon fibre lamellas can be installed after the concreted is hardened by means of special epoxy glue. This technique is used in situations in which the capacity of structures should be increased. It should be noted that the material usually is applied to existing structures that need to be upgraded as a result of changed demands or to solve mistakes in design or execution.

Figure 6-27 Application of carbon fibre lamellas

The application of external carbon fibre reinforcement is not commonly applied. There exists no design code for this method in the Netherlands, there is however a guideline, CUR recommendation 91, available which can be used.

The fiber lamellas are prefabricated by means of pultrusion. In a continuous process, carbon or fibers are drenched in epoxy resin and hardened by means of heating, as indicated in figure 6-28. The thickness of the lamellas is in the order of millimeters.

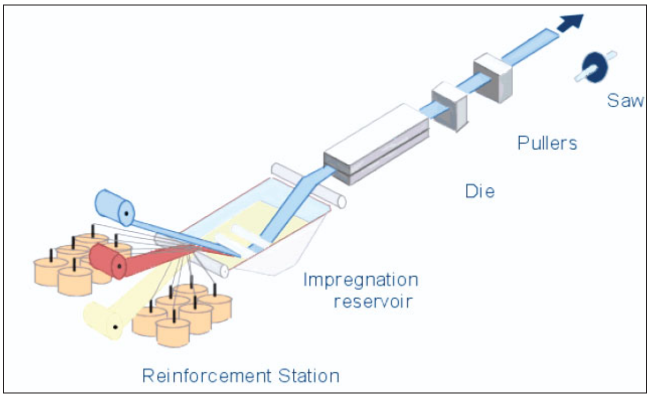


Figure 6-28 Production process carbon fibre lamellas

Characteristics of the material

For this material, there are several manufactures as a result of which the properties of the materials may slightly differ. Therefore, again, the general behavior will be described and typical values for different characteristics will be provided in the following paragraphs. In the table below typical mechanical properties in longitudinal direction are listed, for a fiber content of 40 %. These values will strongly vary with the fiber content and quality of the laminate. However, the order of magnitude for the several properties is representative.

		High strength carbon fiber	High modulus carbon fiber
Tensile strength	[N/mm^2]	1,000-1,900	800-1,400
Compressive strength	[N/mm^2]	~1,000	~600
Modulus of elasticity	[N/mm^2]	100,000-120,000	140,000-240,000
Strain at fracture	[%]	1.5-2.2	0.6-1.4
Density	[kg/m^3]	1440	1480
Expansion coefficient	[$\cdot 10^{-6}/^{o}$c]	0.3	0.3

Table 6-4 Characteristics of carbon fibre lamellas

- The tensile strength of carbon fibre is in the order of 5 till 10 times as high as the tensile strength of steel.
- The durability of the carbon fibre lamellas is no problem, since it is not susceptible to corrosion.
- Carbon fibre is a rather brittle and therefore it should be applied for withstanding the explosion load only, whereas the permanent loads are withstood by means of regular steel rebar.
- Carbon fibre lamellas are approximately 75% lighter than steel, which is advantageous for the installation phase.
- For the bottom of the element the lamellas can not be applied easily. The supporting subsoil provides resistance however and therefore this element is relatively less vulnerable. Besides this, a non reinforced layer of ballast concrete will be present in the final situation increasing the capacity of the element. Therefore it may be possible that applying lamellas at the bottom is not necessary.
- Installing the lamellas at the walls is rather complicated, since extensive scaffolding is required.

- The installation of the carbon fibre lamellas is quite labour intensive, since the surface of the structure should be roughened in order to provide sufficient bond between the lamellas and the concrete.

- An aspect that requires special attention is the resistance to fire of the laminates.

6.6.4.1 Conclusions
The following conclusions can be drawn with respect to the application of external reinforcement.

- The complex and labour intensive execution make the application of carbon fibre lamellas not attractive.

- The application of external carbon fibre lamellas is usually applied for repairing or improving the capacity of existing structures instead of new structures. The method is therefore less applicable in this case.

6.6.5 Alternative 3.5: Fibre reinforced concrete

A rather new technique to improve the structural performance of concrete concerns the mixing short fibres. These fibres will be equally distributed in the mixture and have a random orientation, in this way the material is reinforced homogeneously. Different types of fibres, like steel, glass, synthetic materials as well as natural materials can be used.

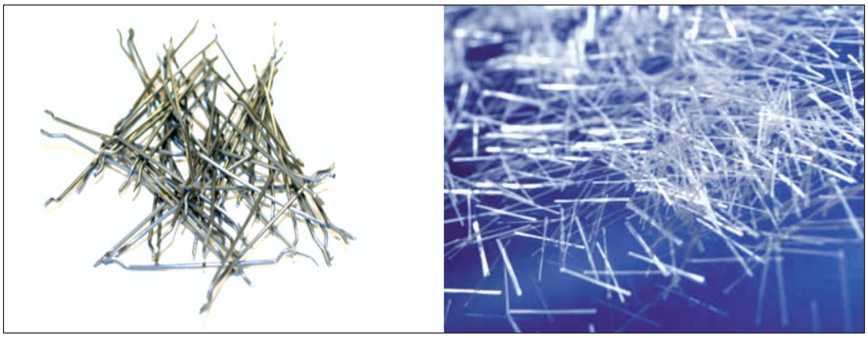


Figure 6-29 Examples of fibre reinforcement

The most important benefits of fibre reinforcement are an increased tensile strength, crack resistance and ductility. These properties are favourable for the resistance against explosions. Providing the same flexural capacity as with traditional reinforcement is however rather progressive nowadays. The reason for this is that traditional reinforcement will be applied at the locations where it is most effective, whereas the fibres are mixed with concrete. In combination with rebar a slightly better capacity could be achieved though this is not considered to result in an efficient solution. The strength of the material hardly increases as a result of the application of fibre reinforcement, so still large thicknesses will be necessary. Besides this, the material is up to five times as expensive as regular concrete.

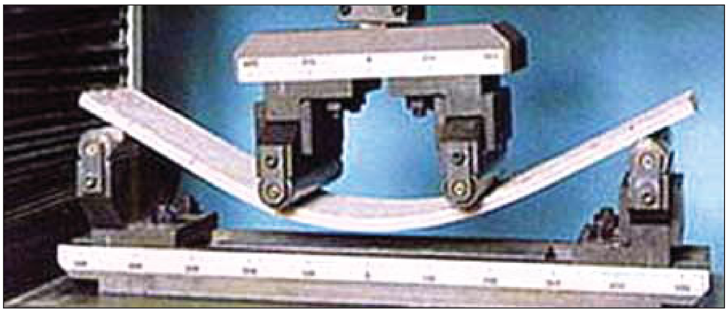


Figure 6-30 Ductile behaviour fibre reinforced concrete

Recently, research related to the blast resistance of this composite material by The Civil Engineering Department at Penn State University is performed. A special combination of fibre reinforcement and traditional steel rebar is studied in this respect and the results are promising, though the research still continuous and application is not possible for now.

There are however other advantages to this material. The application of synthetic fibres improves the resistance against fire. Because of the high temperature, the fibres will melt, while small channels remain. The concrete becomes more porous, and the pressure that develop in the material as a result of the vaporising moist, can escape, which prevents sputter. In case steel fibres will be applied, the fire resistance of the concrete will improve also, though the effect is of less magnitude. The steel fibres will provide a better cohesion of the top layer, which prevents sputter, though the built up of vapour pressure is not limited.

6.7 *Separated tubes*

As stated before, it is preferred by BAM Infraconsult to apply a box shaped concrete cross-section. There could be some variation in geometry and arrangement of the tubes that may be beneficial for the development of an explosion resistant tunnel. If a connection will be used intensively by traffic that transports hazardous goods, it could be considered to facilitate separate tubes for these vehicles. In this way the danger is isolated from the regular traffic. Therefore, the probability of an accident will decrease. Since the special tubes can be of small dimensions, accommodating only one lane in each direction, it is easier to provide sufficient structural capacity in order to withstand an explosion load. An indication of this principle is presented in figure 6-31. Since usually only a limited percentage of the total traffic concerns transport of dangerous goods, the special tubes will probably not be used efficiently. The reduction in casualties as a result of the isolation of the problem may be a motive to apply separate tubes. The focus initially was on the development of an explosion resistant regular cross-section, since this complies with the requirements as usually stated by the client. It should be noticed however that there are important advantages that may justify the application of separate tubes. In case of lower intensities, it could be considered to build only one special tube for which the direction of the flow can be alternated, a sound traffic control is required in that case.

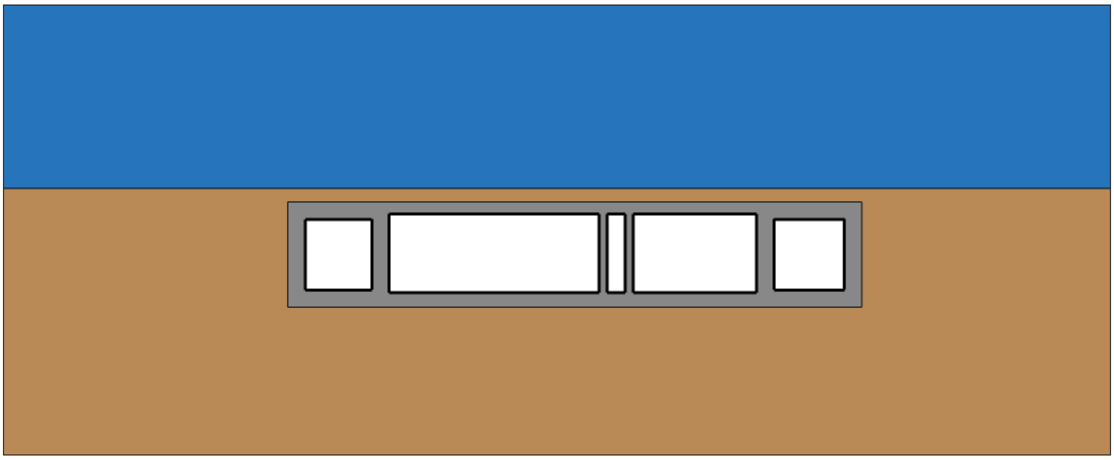


Figure 6-31 Principle of separate tubes

- As a result of the smaller dimensions of a special tube it is easier to provide the required structural capacity.
- The major part of the cross-section can be of regular, efficient dimensions.
- Since the tube will provide one lane that is used by trucks only the probability for an accident will decrease.
- The vehicles transporting dangerous goods are isolated from the regular traffic. Therefore, the casualties in case of an accident will be limited significantly.
- Damage as a result of an explosion will concern only the special tube, the connection will not be out of service for the regular traffic due to an explosion.

- Special exits from the highway should be created as well as provisions for waiting trucks in case just one tube will be applied.

- Trucks that make use of the special tube can be obliged to pay toll. In this way the extra costs involved can be partly covered.

6.8 *Weight limiting measures*

Since rather large structural capacity is required in order to withstand the explosion load, the deadweight of the structure will be large. Since the element has to be transported afloat, this can be a problem. Therefore, it may be of interest to consider weight limiting measures in this respect. In the following paragraphs some ideas are discussed.

6.8.1 Low density concrete

In case the deadweight of the element is of such magnitude that the buoyancy becomes a problem, it could be considered to apply concrete with a lower density. Recently, a special vessel to dismantle rigs at sea was designed in low density concrete. For this purpose extensive research was performed to the mechanical properties and workability of this material and the results are very promising. A light weighing additional material, on basis of clay was used. A strength class of C35 was realised whereas the deadweight, without reinforcement was less than 1600 kg/m³. Due to the reinforcement, the deadweight increases, though still a significant reduction can be achieved.

6.8.2 Hollow spaces in the concrete

In order to limit the deadweight of the elements, it could be considered to create hollow spaces in the structural elements. Since the material is especially required in the outer areas of the cross-section and to a lesser extent in the middle parts. Hollow spaces can be easily realized by casting pipes in the slabs as indicated in figure 6-32. After immersion, the pipes can be injected if desired.

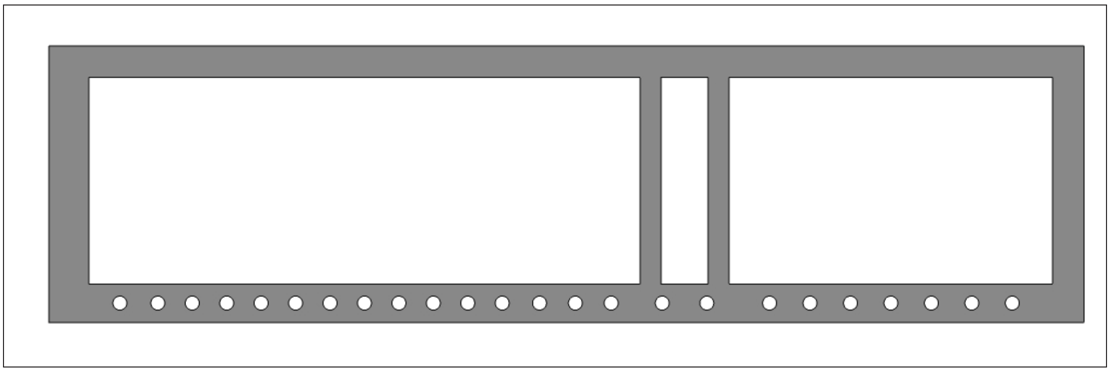


Figure 6-32 Application of hollow spaces in order to reduce the deadweight

A possibility to reduce the deadweight of concrete structures is the application of air filled synthetic spheres. These spheres can be placed between the reinforcement nets, providing hollow spaces. The element deadweight of the floor and roof slab can be reduced significantly in this way.

Figure 6-33 Example of a bubble deck slab

6.8.3 Staged construction

As stated before, during transport the elements should be afloat, whereas in the final situation a high structural capacity as well as sufficient large deadweight in order to ensure stability of the tunnel is required. These are contrary demands. It could be considered to adjust the construction method in order to comply with the demands for the different stages. Since the elements should have a large thickness in order to provide sufficient capacity for withstanding the high explosion load, it may be difficult to design the element in such a way that it can be transported afloat. If the elements initially are build with sufficient structural capacity for the transport and immersion operations. When positioned, the elements can be finished from inside the tunnel. The walls and floor slab may be suitable for the application of this principle. For the walls the execution becomes relatively complicated and it is questionable if this is a suitable solution for these elements. Staged construction of the floor seems more plausible in this respect.

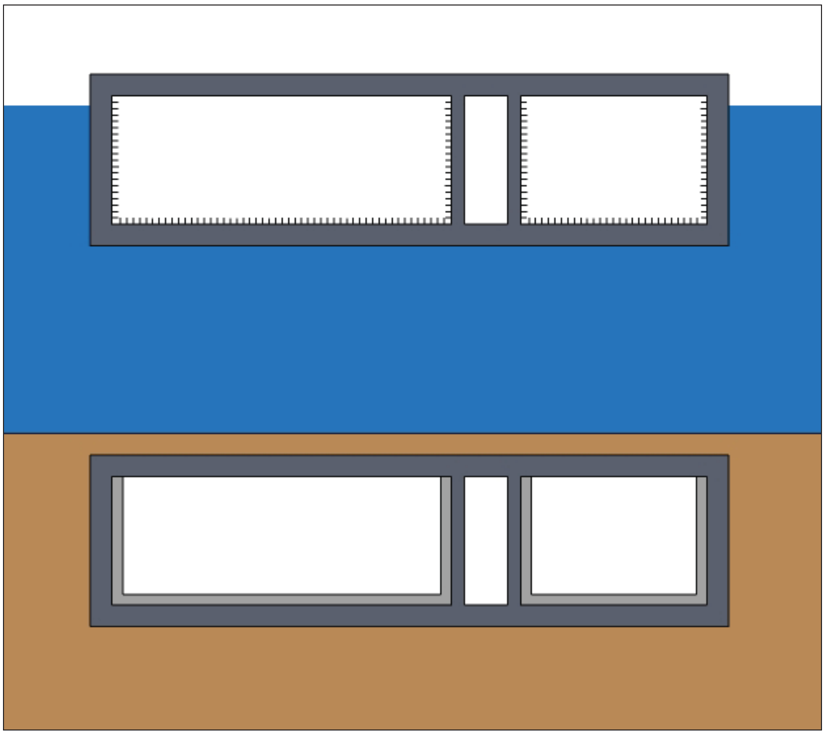


Figure 6-34 Phasing of the execution

An advantage of this method is that the structural concrete that is applied for the finishing is also ballast for the element.

6.9 *Evaluation*

In this paragraph an evaluation will be made concerning the various alternatives. In order to do so, comparison will be made for several aspects. From the developed alternatives, it became clear that not all alternatives provide efficient solutions. Besides this, the effects of an explosion are not equal for all cases and the side effects may differ per alternative. In this stage it is attempted to find a solution that provides as much functional advantage as possible. Therefore, a number of alternatives will be not considered any further on forehand, these are listed below.

- Alternative 1.1 Increasing the cross-section
  From a quick analysis, it became clear that increasing the cross-section seems to provide no efficient solution for the resistance against explosion loads. Therefore this will not be considered any further.

- Alternative 1.2A Weak areas in the roof
  There are a number of important drawbacks connected to this alternative. It is very unlikely that this is a realistic solution. Extensive research would be required and the effect will be dependent on the location to a large extent probably. Within the framework of this research no further attention will be paid to this alternative.

- Alternative 1.2B Weak areas in the intermediate walls
  Safety of people inside a tunnel is an important issue, for that reason dangerous goods are prohibited nowadays. Although there seems to be a trend to except casualties incidentally this should be obviously limited as far as possible. Besides this, both tubes will be damaged instead of one.

- Alternative 2.2 Absorbing coating
  Application of an absorbing coating is not considered to be a realistic solution, since the loads are of severe magnitude the support force on the structure will still be of a large proportion. Besides this, the materials that are potentially suitable to be applied as coating are expensive. Therefore, it is decided to not consider this alternative any further within the framework of this research.

- Alternative 3.4 Application of external reinforcement
  The application of external reinforcement is rather complex and more suitable as method for repairing existing structures than for reinforcing new ones.

- Alternative 3.5 Application of fibre reinforced concrete
  If fibre reinforced concrete is applied, the structural capacity will be increased. This increasement is very limited however and will not provide a solution. It is possible to apply fibre reinforcement in combination with traditional steel rebar. From recent studies to the blast resistance it became clear that this may be developed to a suitable alternative in the future. There are however advantages with respect to fire resistance and cohesion of the top layer and therefore it could be beneficial to apply this material.

Furthermore, it can be concluded that a few solutions may be suitable locally, providing a partial solution. These are listed below.

- Alternative 1.2 A Cut and cover section without roof
  Building the cut and cover sections of the tunnel without a roof. In this way the design problem reduces to the immersed part of the tunnel since the pressure is released easily in the uncovered entrances.

- Alternative 2.1 Application of secondary walls
  The application of secondary walls may relieve the intermediate and outer walls of the tube. The roof slab should be provided with sufficient structural capacity however.

- Alternative 3.1. Application of a structural floor in the emergency corridor.
  The application of a structural floor has a favourable effect on the structural capacity of the intermediate walls. This adaptation is relatively easy, though it will be only a partial solution that requires additional provisions.

- The weight limiting measures could be considered if the buoyancy of the elements is a problem.

A number of the considered alternatives potentially provide a solution for the design of an explosion resistant tunnel. These are discussed briefly below.

- Alternative 2.3 Sandwich structure
  The application of a second tube inside the tunnel, whereas the space in between will be filled with a material that provides damping and energy dissipation is a promising solution at first glance. Therefore it will be considered further.

- Alternative 3.2 Application of pre stressing
  Applying pre stress can significantly increase the structural capacity of the cross-section. Therefore this may be beneficial.

- Alternative 3.3 Application of fibre reinforced polymer rebar
  Applying synthetic reinforcement as a substitute for the traditional steel rebar seems to be advantageous for requirements comparable to those that were stated for the Oosterweel tunnel lately. Therefore this solution will be considered further within the framework of the research.

- Alternative 4 separated tubes
There are a number of important advantages to this alterative. Besides this, the favourable structural aspects, also a reduction of the risk and an increased level of safety for all users can be achieved.

The most promising alternatives will be judged on a number of aspects, to make a well considered choice. The aspects that are considered in the evaluation are listed below.

- Similarities to traditional concept, since processes are standardized to a large extent it is desirable to find solutions close to the traditional concept.

- Construction phase, the handling of materials and constructability are important aspects for the building time and costs.

- Experience and well known techniques generally result in standardized processes and fewer risks for the execution phase.

- Reduction of risks during life time, although the starting point of the thesis is to accept casualties, prevention is always preferable. Besides this, the risk for the connection to be out of service should be limited if reasonably possible.

- Range of application, the order of magnitude for which an alternative can be applied is of importance. Distinction is made between the applicability of the alternative for the requirements as stated recently for the Oosterweel tunnel and the more extreme representative BLEVE load according to TNO.

In table 6-5 the scores for the several aspects are listed.

Solution	Similarities traditional concept	Construction phase	Experience With techniques	Risk reduction	Requirement Oosterweel	BLEVE load TNO
Sandwich structure	0	-	+	0	-	+
Application of pre stress	0	-	+	0	+	-
Application of FRPR	++	++	-	0	++	-
Separate tubes	+	0	+	++	++	++

Table 6-5 Comparison of the alternatives

From this quick evaluation it can be concluded that the application of pre stress, as well as applying Fibre Reinforced Polymer Rebar are a promising alternatives for requirements concerning explosion loads of the order that were stated for the Oosterweel tunnel. The application of FRPR reinforcement may be beneficial in this respect, some additional information concerning this rather new material can be found in Appendix I. For higher loads, the cross-section has to be increased since the compressive capacity of the concrete becomes critical. An increasing cross-section results in problems with the buoyancy of the elements and should be limited therefore. The application of a sandwich structure may be not the most optimal solution for the requirements as stated for Oosterweel tunnel, the dimensions of the element significantly increase whereas there are other alternatives to provide the required capacity. For the representative BLEVE load according to TNO, the sandwich structure is considered to be beneficial, since the primary tube is relieved to a great extent. Furthermore, the application of separate tubes is advantageous in this respect for many reasons. The structural capacity is relatively easy provided with this solution and the main part of the cross-section can be of regular dimensions. Besides this, since it is determined that requirements for an explosion resistant tunnel should be of considerable larger magnitude than the requirements stated for the Oosterweel tunnel recently, it is decided to investigate this alternative further.

7 Promising alternatives

In this chapter the alternatives that are qualified to be promising and interesting will be investigated further. The alternatives considered comprise the application of a sandwich structure and the application of separate tubes. In order to judge the structural feasibility, exploring calculations are performed. For both alternatives an evaluation regarding the feasibility is made. Besides this, a few structural details that are characteristic for an immersed tunnel are considered, of which the results are generally applicable.

7.1 *Sandwich structure*

The application of a sandwich structure seems to be beneficial and will therefore be considered in more detail. The most important advantages to this alternative are listed below.

- The secondary tube relieves and protects the primary tube, which is not directly exposed to the explosion load.
- Execution can be staged, which is advantageous for complying with the contrary demands in the transport phase and final situation.
- The structural capacity is considered to be increased to a large extent
- In the event of an explosion, the main tube is still well protected against fire in spite of the possibly destroyed fire resistant coating.

7.1.1 Design

For the design of a sandwich structure, there are a few aspects of importance, these are listed below.

- Since the inner tube is not exposed to the loads due to the water and soil pressure, the effect of the rebound on this inner tube will be very small compared to the regular cross-section.
- Unacceptably large deformations or collapse of the primary tube should be prevented. The inner tube may be destroyed completely however.
- It is desirable to limit the thickness of the inner tube and fill material, since the cross-section will be of excessive dimensions otherwise.
- If dissipation of energy via the fill material should be achieved, a considerable thickness is expected to be required. Therefore, a sandwich structure results in a significant increase of the cross-section.
- Supports that connect the inner and outer tube, causing concentrated shear forces should be avoided.
- The fill material should have low strength compared to the outer tube, energy dissipation can be achieved by either compressing or crushing the material.

The inner tube can be replaced after an explosion has occurred. An additional advantage of this alternative is the possibility to construct the tunnel initially without the inner tube. If it is during the lifespan required to make the tunnel explosion resistant, the secondary tube can be applied. The tunnel should be provided with sufficient dimensions to accommodate the tube later in that case.

An important disadvantage to this solution is that the cross-section will be significantly increased. The thicknesses of the primary tube should be at least the same as for the situation in which no explosion loads are taken into account, in order to withstand the water and soil pressure. The thickness of the fill material should be in the order of 1 meter at least as a first approximation. Furthermore, the secondary tube should be provided with sufficient capacity. Therefore, the height as well as the spans of the element will increase significantly, which probably results in larger thicknesses as well. Especially the height of the element is of importance since it has large influence on the length of the structure and therefore it is a very important aspect for the costs of the project. The effect of the increased height may be considerable. A representative maximum value for the slope is 4%. If the tunnel will be situated 1 meter deeper, this results in an additional length of 25 meter for both entrances. The fill material and secondary tube contribute to the deadweight, which makes the application of ballast concrete unnecessarily. The height of the elements will be increased however, since the height for ballast concrete is usually in the order of 1 meter only. In the figure below, these considerations are graphically presented.

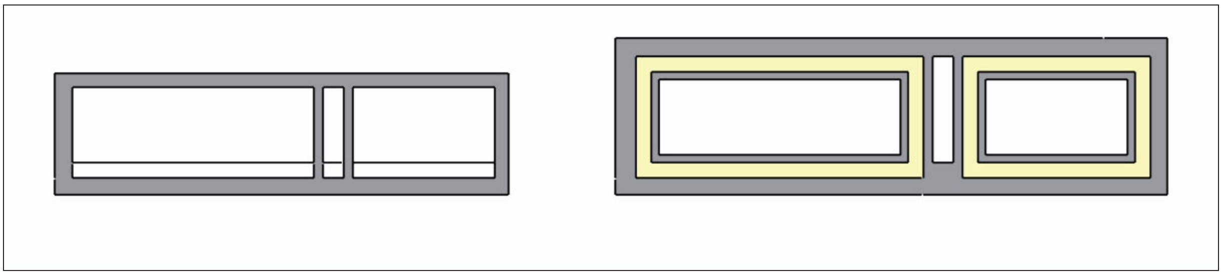


Figure 7-1 Comparison between the dimensions of a regular cross-section and a sandwich structure

Since the floor is relatively less vulnerable to the explosion load compared to the other elements, it could be considered not to apply the fill material here, which will reduce the height.

7.1.2 Elaboration

In order to investigate the technical feasibility of this alternative, exploring calculations are performed. These calculations comprise the stability of the element, both during transport and in the final situation. Furthermore, the resistance against a representative explosion is considered and the order of magnitude for the required dimensions is determined. Apart from that, the influence of the properties of the fill material is investigated. For the evaluation, the dynamic module of Plaxis is used. The results of the calculations can be found in Appendix J.

In the table below, the main dimensions and quantities for the regular cross-section and the sandwich structure are listed. It should be noticed that the sandwich structure is not optimized and therefore the numbers just give an indication. The quantities for the smaller tube could be less, therefore the amount of concrete is overestimated. It can however be concluded that the required amount of structural concrete would be more than twice the required amount for a regular cross-section.

		Regular cross-section	Sandwich structure
Width overall	[m]	29.9	36.35
Height overall	[m]	7.9	10.9
Structural concrete	[m²]	81	150+40 =190

Table 7-1 Indication main dimensions sandwich structure

- Dimensions of the primary tube should be increased to a large extent compared to a regular cross-section.
- The influence of the compressibility of the fill material is very limited.
- The amount of reinforced concrete increases excessively.
- The height of the elements is increased with 3 meters, which results in considerably longer entrances.
- Due to the increased overall dimensions, the volume that has to be dredged will be increased significantly.
- The dimensions of the element will be increased excessively due to the required space for the sandwich structure.

- The effect of a sand fill is very small, while relatively large space is required to facilitate it.
- The effect of applying a relatively compressible material appears to be very limited as well.
- Due to the large spans of the floor and roof slab large thickness of the secondary tube is required.
- The requirements concerning stability during the transport and final situation are easily met, due to the possibility of staged construction.

It can be concluded that the cross-section has to be increased excessively to provide sufficient capacity. The height of the elements has to be increased considerably, which is unfavourable since it results in long entrances. Apart from that, the extra amount of reinforced concrete that is required is also very large. For larger spans, the disadvantages are of course of more importance.

7.1.3 Execution

The application of a sandwich structure has important influence on the execution phase. The primary tube can be built according to the regular methods, though the secondary tube and application of the fill material require special attention.

The secondary tube can be (partly) constructed after immersion of the primary tube. This is favourable with respect to the contrary requirements during transport and the final situation. Building the secondary tube will result in a significantly increased construction time. The secondary tube can be assembled from prefabricated elements, or completely built in situ, depending on the precise solution and used materials. The application of the fill material can be more complex, besides this, the method for application depends on the type of material. Pumping the material into the cavity could be an option for example.

7.1.4 Use of a tunnel facilitated with a sandwich structure

In the event of an explosion, the interior and secondary tubes are expected to be destroyed. Repair of the tubes will take months. Since the regular traffic makes use of these tubes this is a drastic consequence.

7.1.5 Conclusion

From the evaluation it can be concluded that it is technically feasible to apply this solution. The effectiveness of the principle is however questionable. The alternative requires large amounts of materials. Besides this, additional dredging is required and the execution phase will take significantly more time. In the event of an explosion, many casualties are to be expected and the tunnel will be out of service for months. Application of different materials for the secondary tube and fill material may result in a more beneficial solution, though the effect will be limited.

7.2 *Separated tubes*

Since the occurring pressure is very high and a lot of damage and casualties are to be expected in case of an explosion, the application of a separate tube will be investigated further. The most important advantages are listed below.

- Since the tube will provide one lane that is used by trucks only, the probability for an accident will decrease significantly.
- The vehicles transporting dangerous goods are isolated from the regular traffic. Therefore, the casualties in case of an accident will be limited.
- Damage as a result of an explosion will concern only the special tube, the connection will not be out of service due to an explosion for the regular traffic.
- As a result of the smaller dimensions of a special tube, the required structural capacity is relatively easy to provide.

- The major part of the cross-section can be of regular, efficient dimensions.
- Special exit lanes from the road should be created as well as provisions for waiting trucks in case just one tube will be applied.

7.2.1 Design

From a structural point of view, a separate tube is especially beneficial because the spans can be limited. In order to find an efficient solution, the required free space will be considered in this paragraph for several lay outs.

Required width

The required width of the road is composed of several aspects. Some dimensions are dependent of the design speed of the road. A design speed of 80 km/h seems plausible. For this speed, the dimensions listed in the table below are recommended in [23]. The maximum dimensions for a truck are presented in figure 7-2.

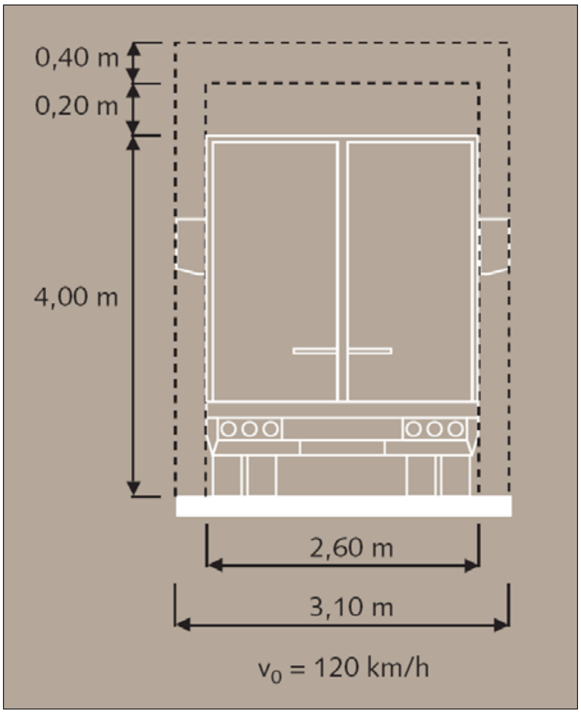


Figure 7-2 Required profile for trucks [23]

Function	Recommended width
Lane	3.35 m
Emergency lane	3.15
Side line	0.20 m
Partition line	0.15 m
Redress line	0.30 m

Table 7-2 Required width

Required height

The required height for trucks is 4.6 m, as indicated in figure 7-2. This value is based on a design speed of 120 km/h. For lower design speeds, the height can be reduced somewhat. There is however additional height required for facilitation of installations. Therefore it is decided to apply a height of 6 m.

The space that is reserved for ballast concrete in a regular cross-section can be completely or partially used for structural concrete in order to provide sufficient structural capacity.

7.2.1.1 Variants

1 lane

In order to limit the spans as much as possible, it could be considered to apply just one lane. This is very unusual, though the application of a special tube is also exceptional. Since no emergency lane is applied, in case of an accident it will be difficult for emergency services to get there. The probability of an accident is however considerably lower, since there is only limited traffic flow and just one lane. From a structural point of view this solution is advantageous, since the span is limited as much as possible.

1 lane + emergency lane
The application of an emergency lane is preferred. It may for instance occur that a truck breaks down inside the tunnel and a tow truck is required to move the vehicle. In that case the tow truck should be able to pass the broken truck. Due to the application of an emergency lane, the total required width increases significantly, as indicated in figure 7-4.

2 lanes
It could be considered to apply 1 lane in both directions, separated by means of a barrier as indicated in figure 7-3.

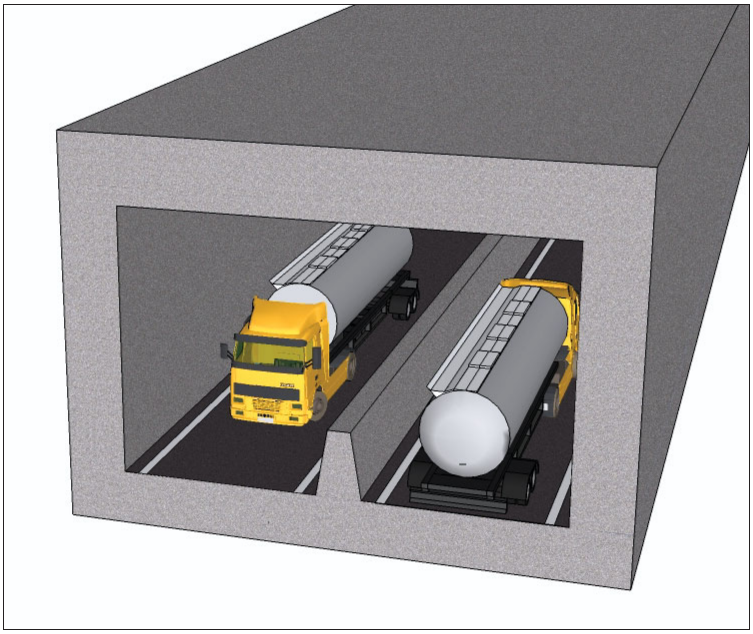


Figure 7-3 principle of two lanes

Special provisions could be applied in order to make it possible to access the other lane for a tow truck or emergency service at certain spots. In that case it can be justified not to provide an emergency lane. The required width increases further however. The spans are of such magnitude that the structural benefits become of less importance.

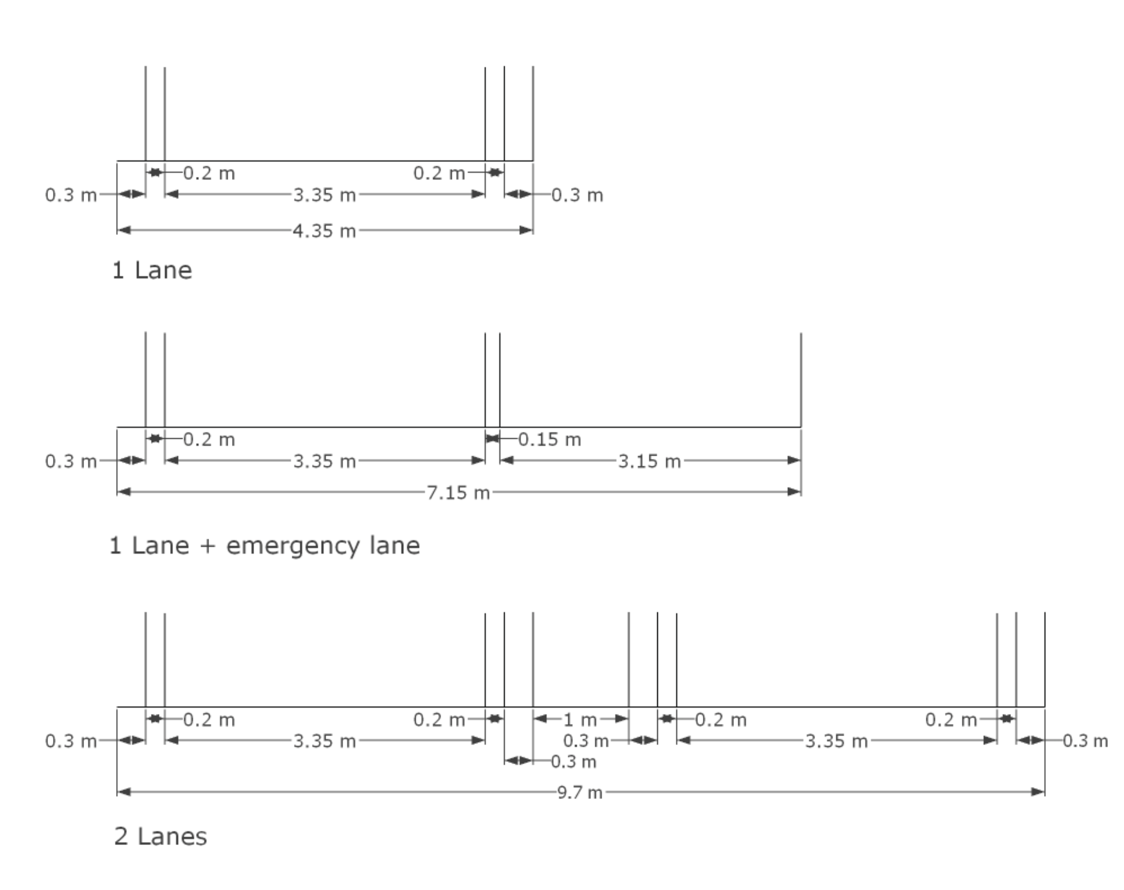


Figure 7-4 Required widths for the different options according to [23]

As stated before, limitation of the span is desirable. The application of a emergency lane is however inevitable in order to provide a suitable solution. Since this concerns a special situation, the width could be reduced somewhat, which can be supported by the following considerations.

- The tube will be exclusively used by qualified drivers.
- The space that is required to pass a stranded truck at low speed can be less.

In order to check the feasibility of this solution, a width of 7 m will be applied keeping in mind that a further reduction may be justified.

7.2.1.2 Manner of separation

Separation of the dangerous transport from the remaining traffic can be achieved by constructing two tunnels. Alternatively a special tube could be attached to a regular cross-section. Both principles will be discussed in the following.

Separate tunnel

Separate tunnels for the regular traffic and dangerous transport can be built, as indicated in figure 7-5.

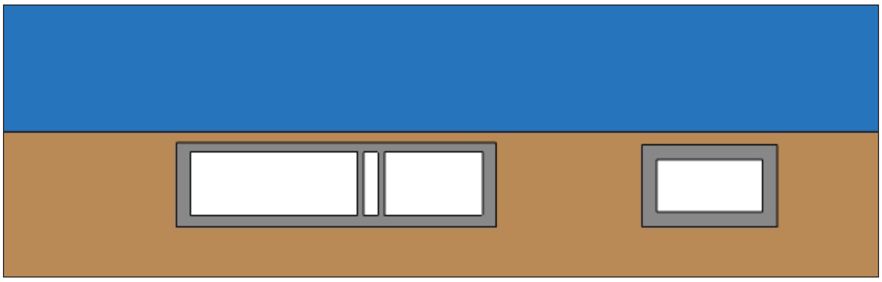


Figure 7-5 Seperate tunnel

There are a few aspects that are of interest. These are listed below.

- The tunnel for regular traffic can be composed of efficient dimensions.
- The fire resistant coating of the regular tunnel has to be designed for small fires only.
- There are no intermediate walls, which are vulnerable elements if it comes to resistance against explosions.
- A disadvantage to this solution is that the amount of material that should be dredged increases significantly and the immersion operation has to be performed twice.

Apart from separation of the dangerous transport from the regular traffic, it could be considered to separate trucks and cars with an excessive height. Besides the above mentioned aspects, there are a few things that are of interest to this idea.

- The height of the tunnel for cars can be considerably smaller, which results in shorter entrances.
- The tunnel for cars can be provided with less capacity
- A better flow is to be expected trough the regular tunnel, since the relatively slow trucks do not use it.
- Since more vehicles make use of the special tube, the risk of an accident increases compared to the situation in which just dangerous transports make use of it.

Separate tubes

It could be considered to attach a special tube to a regular cross-section as indicated in figure 7-6.

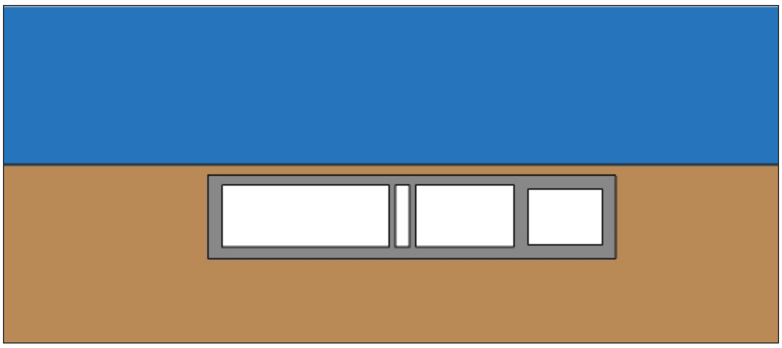


Figure 7-6 Separate tubes

The following aspects are of importance in this respect.

- The major part of the tunnel can be of regular dimensions.
- The regular tubes are indirectly loaded in case of an explosion, though it is expected that the order of magnitude of these effects is only small.
- The major part of the tunnel can be designed for smaller fire loads.
- The intermediate walls between the special tube and the regular tubes should be of large dimensions.
- The cross-section can be designed in such a way that no ballast concrete will be applied in the special tube, additional height is available for structural concrete.

- Due to integration of the different tubes, dredging, construction of the entrances, building of the elements and the immersion operations have to be performed only once, which is efficient.

- For implementation in the environment, it is preferable to have one tunnel.

It is decided to investigate the application of a separate tube with one lane and an emergency lane as well as the application of a special tube accommodating two lanes further. In order to investigate the feasibility of the alternative, the order of magnitude for the required dimensions will be determined.

7.2.2 Elaboration

For both variants the dimensions are determined whereby the mass spring model is used in order to provide estimation for the required thicknesses. The stability during transport as well as for the final situation is checked. The same requirements and starting points as for the regular cross-section except for the explosion load are applied, reference is made to paragraph 5.2. The results of the calculations are attached in Appendix K.

From the exploring calculations it can be concluded that the application of a special tube is technically feasible. The cross-section can be of reasonable dimensions and the requirements for the transport phase can be met. In order to give an indication of the impact on the design, the main dimensions and the approximately required amount of reinforced concrete for the several options is presented in the table below. It should be noted that the numbers are based on exploring calculations. Detailed design and optimization may influence the results. The order of magnitude and proportions are however of interest.

Variant	Height [m]	Width [m]	Concrete [m³/m]	Fly over required
Regular cross-section	7.9	29.9	81	No
Sandwich structure	10.9	36.35	150 + 40	No
1 special tube	8.3	37.75	114	Yes
2 special tubes	8.2	46	140	No
1 special tube, 2 lanes	9.2	40.45	137	Yes

Table 7-3 Comparison variants

It can be concluded that in case 1 lane is required for both directions it is preferable to apply 2 tubes. The reason for this is especially the larger height of the elements, which will result in the need for long entrances. Besides this, in case one tube is applied for two directions, at one side of the waterway, the vehicles should be able to switch to the left side. It is expected that relatively high costs are involved with the required infrastructure. It should be noted that the required amount of reinforced concrete for only the primary tube of the sandwich structure (150 m^3) exceeds the total required amount of reinforced concrete for the application of two special tubes (140 m^3). For the application of special tube with one lane and an emergency lane, more detailed calculations concerning the required amount of reinforcement were made, the results of this calculations can be found in Appendix K and are similar in case for both directions such a tube would be applied.

7.2.3 Execution

The effect of the application of special tubes on the execution phase is only limited. For the construction of the elements there are no special techniques or different approaches required compared to a regular tunnel project. The cross-section will be increased and therefore the execution will take more time. In case one tube for both directions will be applied, a fly over is required at one side in order to enable vehicles to switch to the left side. The required infrastructure will result in an increased construction time. Furthermore, this will result in deviant activities compared to a regular project.

7.2.4 Use of the tunnel with special tubes

With regard to the use of a tunnel that is facilitated with special tubes a few aspects are of importance. These will be discussed in the following.

Consequences of an explosion
In the event of an explosion in a special tube, the traffic in the regular tubes will be safe. The interior of the special tube will be completely destroyed probably, though the structural integrity will not be endangered. Obviously, the entire structure will be checked for malfunctioning after an explosion. After a couple of days, the regular tubes can be used again while the special tube will be repaired.

Traffic management
Obviously, the costs involved with this solution are relatively high compared to a regular tunnel. In order to make the solution more attractive in an economical sense, there are possibilities concerning the use of the tubes.

- It is possible to apply one special tube of which the direction of the flow can be alternated, a suitable traffic light system is required in that case.
- During the rush hours, it could be decided not to allow transport of dangerous goods. The special tubes could be used by regular traffic to increase the capacity of the connection. Since the number of lanes is determined by the peak flow during the rush hours, it may be possible to decrease the number of lanes in the tubes for the regular traffic due to this measure.
- It could be considered to exclusively allow public transport to make use of the special tubes during the rush hours.
- The special tube could be used by emergency services in order to pass the tunnel fast, since the intensity of traffic is expected to be very low.
- It could be considered to obligate all trucks to make use of the special tube. Since trucks are relatively slow, this measure would be advantageous for the flow of the regular traffic. It may be possible to reduce the number of lanes for the regular tubes as a result.

7.2.5 Conclusions

The application of a separate tunnel as well as a separate tube that is attached to the regular tubes both have important benefits for the regular traffic.

- Since the vehicles that transport hazardous goods are no longer mixed with the remaining traffic, the risk for the regular tubes will reduce.
- The regular tubes can be of efficient dimensions.
- Attaching special tubes to a regular cross-section is preferred since all processes and activities have to be performed only once.
- Limitation of the span is the most important advantage to the separation of the tubes from a structural point of view.
- There are several possibilities to make the alternative more attractive in an economical sense.
- This alternative is considered to be a suitable solution for an immersed tunnel that enables the passage of dangerous goods.

Economical feasibility

Although this thesis emphasis on the structural aspects and technical feasibility of an explosion resistant immersed tunnel, the economical aspects are decisive for the actual realisation. The most relevant aspects in this respect are listed below.

Costs

- Additional construction materials.
- Additional labour.
- Additional dredging.
- Increased construction time.
- Special exit lanes of high way.
- Slightly increased length of the entrances.

Benefits

- Shorter routes for dangerous goods.
- Minimal risk for users of the tunnel.
- Potential danger isolated, effects limited.
- Structural safety provided.
- Smaller fire load for the regular tubes.
- Possibilities for other functions (special tubes).
- Levy of toll.

If the application of special tubes is beneficial, is strongly dependent on the local circumstances, intensity of transport of dangerous goods and desired degree of safety.

7.3 *Points of attention*

Apart from the overall strength and stability of the tunnel structure, special attentions should be paid to details. In the event of an explosion, local weak spots may result in additional casualties and damage. Extremely, the structural integrity of the tunnel may be endangered. In this paragraphs a few aspects that are relevant in this respect will be discussed briefly. The purpose is to recognize the problem more than to provide a detailed solution.

Emergency doors

In case of an emergency, people should be able to flee from the isolated tunnel tube. Therefore, a emergency corridor is usually present, which is accessible from the traffic tubes via special doors. In case if an explosion, these emergency doors are weak spots. As a result of the high pressure that suddenly occurs, the doors may be blown out, endangering the corridor as well as the second tube. In case an explosion resistant tunnel is required, attention should be paid to the blast resistance of the doors therefore.

Notches

There are facilities that are usually applied in notches, in order to achieve a smooth finish of the walls. Examples of these facilities are emergency phones and aid posts containing a fire extinguisher as indicated in figure 7-7.

Figure 7-7 Aid post in notch

These notches reduce the structural capacity of the walls however, which may result in local failure in the event of an explosion. The emergency corridor or even the second tunnel tube may be endangered resulting in additional damage and casualties. Facilities like these should be designed in such a way that local failure is prevented in the event of an explosion.

Expansion joints

In order to prevent large stresses due to fluctuations of the temperature or as a result of unequal settlements, expansion joint are usually applied every 18 – 25 meter. A suitable solution for this type of joint is the application of a shear key as indicated for the walls in the figure below. Between the elements, a steel-rubber joint profile is applied, to ensure a watertight connection.

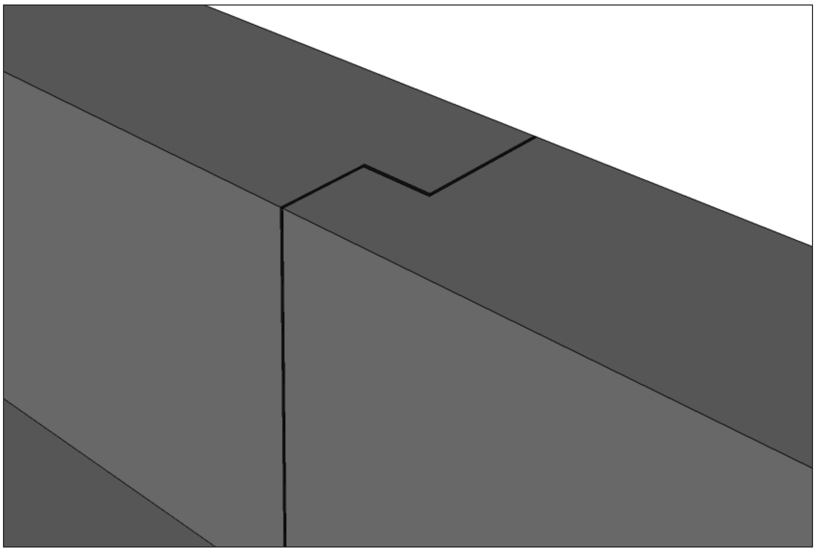


Figure 7-8 Expansion joint

In the event of an explosion, a large shear force may develop, endangering the structural integrity of this connection. It should be investigated whether the capacity is sufficient or not. In case the capacity is insufficient, a solution may be found in the application of large amounts of reinforcement locally. Alternatively, a protective steel plate at the inside of the tunnel could be beneficial. The plate should be connected in such a way that expansion and small rotations of the elements are possible. This could be achieved by means of a sliding connection. An indication of a possible solution is depicted in figure 7-9.

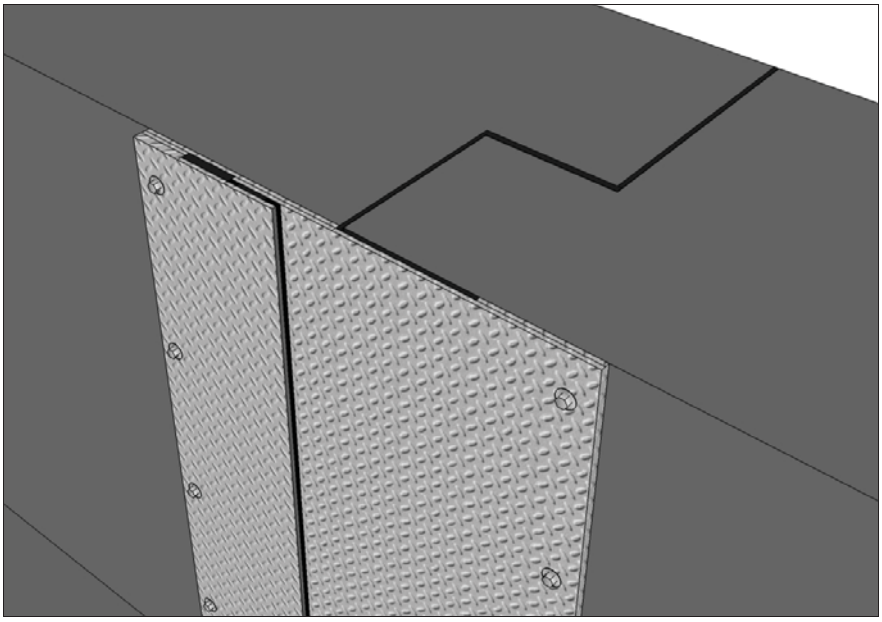


Figure 7-9 Indication of protection for expansion joint

Immersion joints

In order to provide a watertight seal between the elements, special profiles are applied at the end of the element at the entire circumference of the cross-section. As a result of the water pressure, the tunnel elements are pushed against the others. The Gina profiles are made of rubber and will squeeze as a result of the immersion force, providing a suitable watertight seal. After immersion, the Gina profile measures approximately 100 mm. Besides the Gina profile, a secondary seal is applied at the inside of the tunnel, the Omega profile. This secondary seal is composed of a rubber slab that is reinforced with canvas. It is connected to the concrete elements by means of clamping strips.

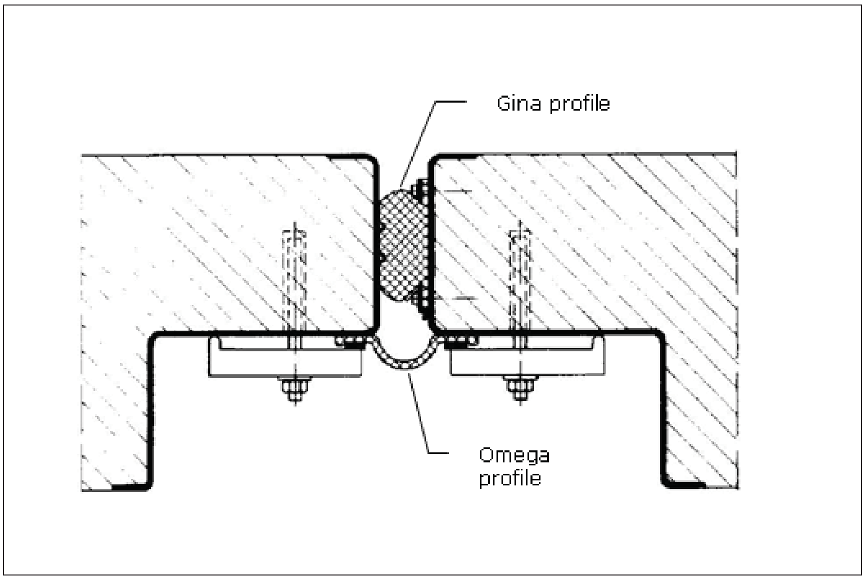


Figure 7-10 Immersion joint, edited from [22]

The joint is usually covered with rock wool and sprayed concrete, as indicated in the figure below.

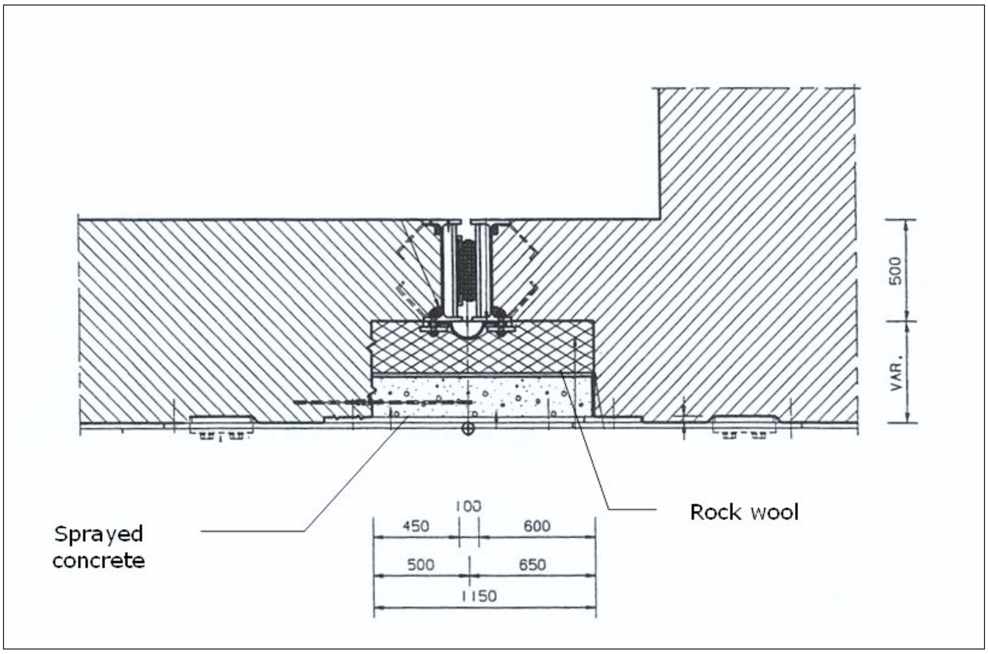


Figure 7-11 Regular finishing of an immersion joint, edited from [22]

In the event of an explosion, a high outward directed pressure will occur. The joints between the elements are weak spots in this respect. Therefore, special attention should be paid to the capacity of the connection, in order to prevent the seals to be destroyed, resulting in leakage or instability of the structure. In order to determine if a problem has to be expected, detailed research is required. This is beyond the scope of this research, though some considerations will be discussed briefly.

- As a result of the immersion force, there exists a pre stress in the Gina profile which is favourable for the resistance against an explosion load.

- Since the span of the joint is rather small, it should be achievable to provide sufficient capacity.

- It could be considered to apply a blast resistant cover plate as protection for the joint since the span is only limited. Steel plates, connected to the elements by means of nuts can be suitable in this respect, as indicated in figure 7-12 . The diameter of the holes in the plates should be larger than the diameter of the nuts, in order to enable small displacements and rotation of the elements. Alternatively a sliding support similar to the solution presented in figure 7-9 could be applied.

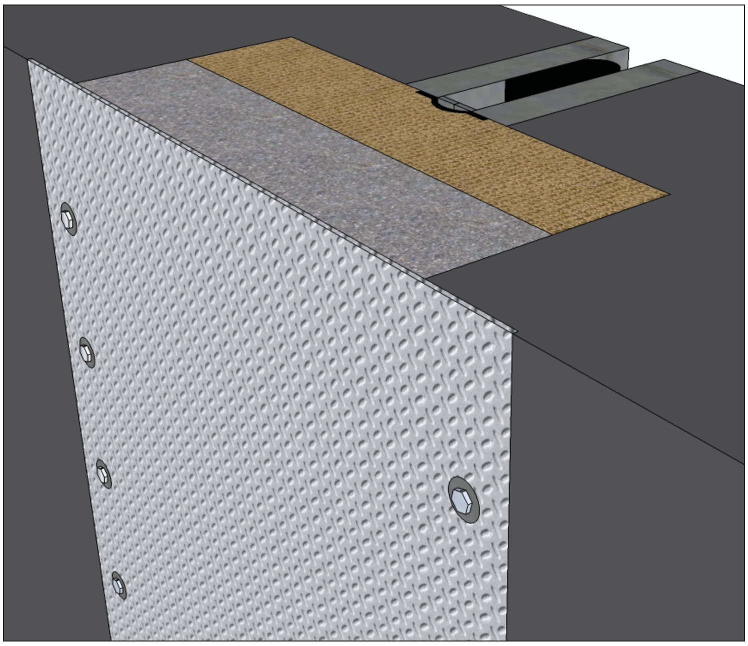


Figure 7-12 Indication of the application of protective plates

- Since the explosion load is unidirectional, it is not to be expected that a complete element will move with respect to the remaining tunnel in the event of an explosion.

Fire resistant coating

As a result of an occurring explosion, the fire resistant coating will be probably destroyed at a considerable length. Since it is very likely that an explosion is followed by a severe fire, which endangers the structural integrity, this could be problem.
It could be considered to improve the fire resistance of the concrete by means of the application of a layer of fibre reinforced concrete as discussed in paragraph 6.6.5. Besides this, regular fire protection should be applied. In case of a fire, the structure is protected by the regular fire resistant coating. In the event of a fire in addition to an explosion, it is expected that the regular coating is destroyed, and the layer of fibre reinforced concrete provides protection. A disadvantage of this solution is that this will provide protection against a fire in addition to an explosion once. Since the probability for such an event is rather low, this may be acceptable though.
This aspect should be investigated further since it is of major importance for the performance of the structure

8 Conclusions and recommendations

This chapter provides an overview of the most relevant conclusions resulting from the performed research. The objective for this research is dual, since it concerned the evaluation of the explosion load as well as the investigation of solutions with respect to the structural design of an explosion resistant immersed tunnel. Because of this duality, distinction is made for the conclusions as well. For both parts of the research a general conclusion is formulated, supported by several more specific statements. Apart from the conclusions, a few recommendations are stated with respect to the applicability of the results and further research. Furthermore, the status of this report will be discussed.

8.1 *Conclusions regarding the evaluation of the explosion load*

The statically defined requirement concerning explosion loads as stated for the Oosterweel tunnel recently, does not comply with the representative explosion according to TNO, since the order of magnitude is too small by far.

- From the literature study it can be concluded that there are no clear and applicable requirements concerning explosion loads stated in the design codes that apply for the Netherlands. These complex phenomena are investigated by TNO, of which the results can be used.

- From recently performed research by TNO, it is concluded that the representative explosion load is due to a LPG BLEVE.

- With the analytical mass spring model the response of structural elements to an explosion load can be modelled. Results comparable to those obtained in research by TNO, using a finite element modelare found. The mass spring model can be used as a screening tool and to investigate the order of magnitude of effects due adaptations of the design.

- Schematizing an explosion load by means of a static requirement is not a suitable approach. Since response of a structure to an explosion load is dependent on the structure itself it is not clear what safetylevel is required if a static requirement is stated.

- The finite element code Plaxis is a suitable tool to investigate the response of an immersed tunnel to the dynamic load, taking into account the effects of the soil and water.

8.2 *Conclusions with respect to the technical feasibility study*

From a structural point of view it is possible to design an immersed tunnel that has sufficient capacity to withstand the representative explosion load according to TNO. Based on the findings, the application of separate tubes for transport of dangerous goods is considerd to be most efficient in this respect.

- Although the static requirement that is stated for the Oosterweel tunnel is of a too small magnitude, it has a strong influence on the structural design and the delicate balance that exists between the construction phase and final situation.

- Providing a regular reinforced concrete cross-section with sufficient capacity to withstand the representative explosion load according to TNO, results in excessive dimensions if even achievable. The requirements for the transport phase can not be met reasonably.

- The large spans of the tubes are the major problem to achieve sufficient capacity efficiently.

- It is technically possible to provide sufficient structural capacity by applying a sandwich structure. Furthermore, staged construction can be applied which is favourable for the interaction between the transport phase and final situation.

- A sandwich structure results in large dimensions and required amounts of material, which makes the alternative less attractive in an economical sense.

- Applying special tubes for the vehicles that transport dangerous goods is technically feasible and complies very well with the traditional concept of an immersed tunnel.

- The risks involved with transport of dangerous goods trough the tunnel will decrease in case separate tubes are applied, both the probability and the consequences reduce.

- In an economical sense, the application of special tubes is considered to be the most efficient solution for an immersed tunnel that should have sufficient structural capacity in order to withstand the representative explosion load according to TNO.

- If it is economically feasible to provide a tunnel with sufficient capacity to withstand the representative explosion depends on the local circumstances and intensity of the transport of dangerous goods.

- There are possibilities for secondary functions of the special tubes that make the alternative more attractive.

8.3 *Recommendations*

- An explosion load is a dynamic phenomenon and should not be described by a statically requirement as input for a design.

- Since the number of vehicles that transport dangerous goods is only limited, adaptations in the design of those vehicles in order to decrease the probability for an explosion as well as the consequences may be beneficial. Therefore, it is recommended to investigate the technical possibilities and economical aspects involved with this measure.

- It should be determined what normative dynamic load as a result of a BLEVE should be taken into account for the design of tunnels for which dangerous goods are allowed. Currently, the design codes provide no clarity about this.

- In case an explosion resistant tunnel is required, it is recommended to consider the application of special tubes.

- For the calculations and schematization of the tunnel a number of simplifications are applied, it is recommended to perform more advanced calculations in order to investigate the presented solutions.

- The application of special tube should be considered in more detail in order to provide a more optimal solution.

- The aspects concerning some details, indicated in paragraph 7.3 should be investigated in more detail, since these are of major importance for the structural performance of the tunnel.

- In case it is decided to apply special tubes, it is recommendable to investigate the possibilities for additional functions, in order to make the solution more attractive in an economical sense.

- In general the results presented in this report are applicable for cut and cover tunnels as well.

8.4 *Structure of the research*

The structure of the research is schematically presented in figure 8-1 The most important processes, conclusions and decisions are indicated in order make the procedure, relations and coherence between the several aspects clear.

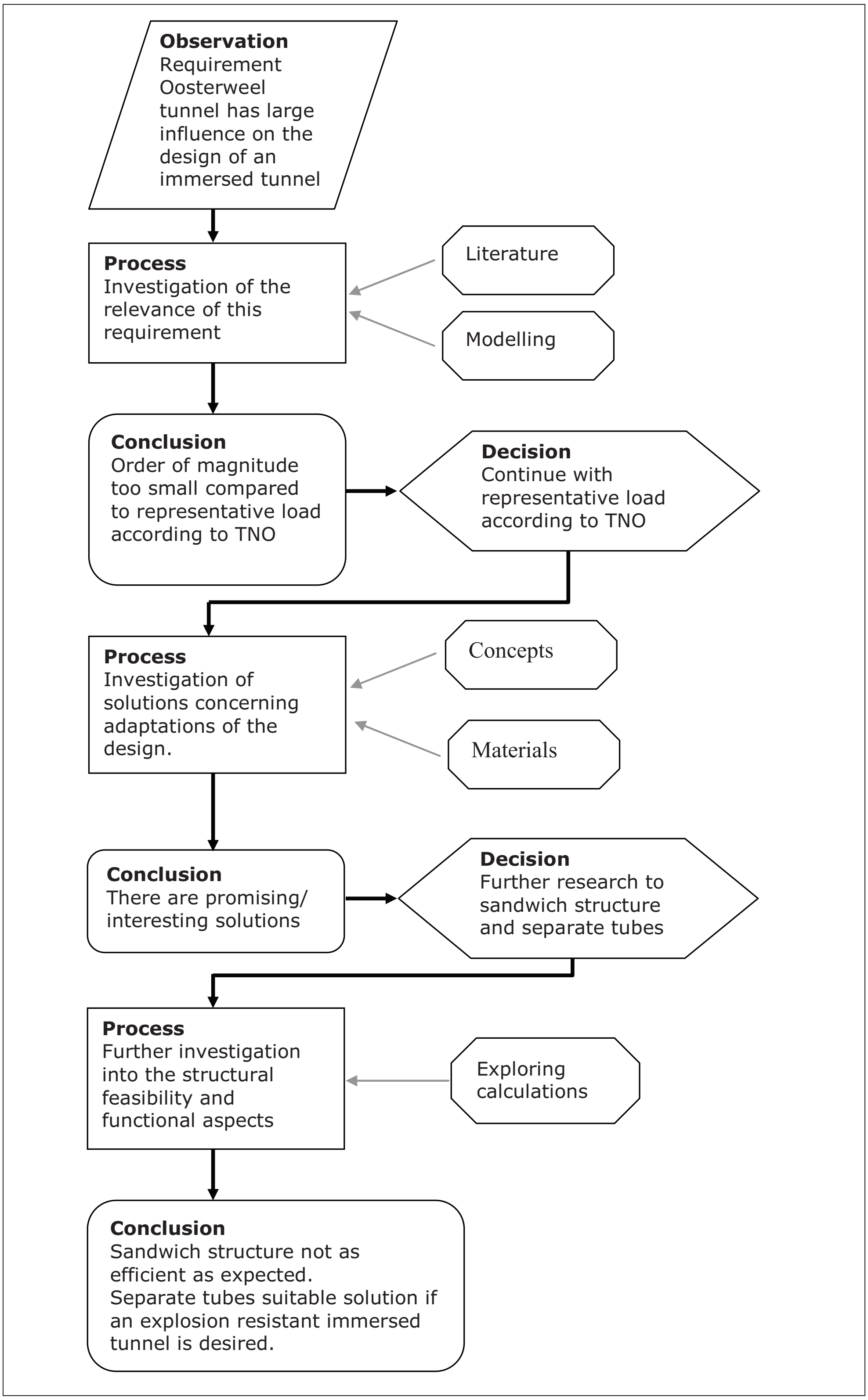


Figure 8-1 Flow chart thesis

8.5 *Status of this report*

The results presented in this report originate from an exploring study. Therefore, it should not be considered to be a complete representation of all aspects involved. More detailed investigation is required in order to verify the obtained results. The major aspects of the problem are considered however and the research contributes to the understanding of the rather complex phenomena. The results provide a sound guidance for future investigation into this topic.

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9.3 *Experts*

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[32] Dr. Ir. J. Weerheijm, TNO and TU Delft

[33] Ir. G.M. Wolsink, Dienst Infrastructuur Rijkswaterstaat

Appendices

Appendix A Maple sheet mass spring model

The code of the Maple sheet that is developed to perform calculations for the mass spring system is printed below. Several sheets are used for evaluation, these have all similar structure. Therefore it is decided to provide only one sheet in order to make principle clear.

```
> restart:
```

Material characteristics

```
> rho:=2500:
> E:=31000000000:
```

Explosion

```
> P0:=500000:
> td:=0.05:
```

Roof

```
> br:=1:
> hr:=1.2:
> Ir:=(1/12)*br*hr^3:
> lr:=14.5:
> kr:=(384)*((E*Ir)/lr^4):
> omega_r:= 22.4*sqrt((E*Ir)/(rho*hr*lr^4)):
> T_r:=(2*PI)/omega_r:
> ur1(t):=1000*((P0/kr)*(1-(t/td)-cos(omega_r*t)+(1/(omega_r*td))*sin(omega_r*t))):
> ur2(t):=1000*((P0/kr)*((sin(omega_r*t)/(omega_r*td))-(sin(omega_r*(t-td))/(omega_r*td))-
cos(omega_r*t))):
> ur(t):=piecewise(t<td,ur1(t),t>td,ur2(t)):
```

Floor

```
> bf:=1:
> hf:=1.5:
> If:=(1/12)*br*hf^3:
> lf:=14.5:
> kf:=(384)*((E*If)/lf^4):
> omega_f:= 22.4*sqrt((E*If)/(rho*hf*lf^4)):
> T_f:=(2*PI)/omega_f:
> uf1(t):=1000*((P0/kf)*(1-(t/td)-cos(omega_f*t)+(1/(omega_f*td))*sin(omega_f*t))):
> uf2(t):=1000*((P0/kf)*((sin(omega_f*t)/(omega_f*td))-(sin(omega_f*(t-td))/(omega_f*td))-
cos(omega_f*t))):
> uf(t):=piecewise(t<td,uf1(t),t>td,uf2(t)):
```

Outer wall

```
> bo:=1:
> ho:=1.15:
> Io:=(1/12)*bo*ho^3:
> lo:=5.97:
> ko:=(384)*((E*Io)/lo^4):
> omega_o:= 22.4*sqrt((E*Io)/(rho*ho*lo^4)):
> T_o:=(2*PI)/omega_o:
> uo1(t):=1000*((P0/ko)*(1-(t/td)-cos(omega_o*t)+(1/(omega_o*td))*sin(omega_o*t))):
> uo2(t):=1000*((P0/ko)*((sin(omega_o*t)/(omega_o*td))-(sin(omega_o*(t-
td))/(omega_o*td))-cos(omega_o*t))):
> uo(t):=piecewise(t<td,uo1(t),t>td,uo2(t)):
```

Intermediate wall

```
> bi:=1:
> hi:=0.5:
> Ii:=(1/12)*bi*hi^3:
> li:=5.44:
> ki:=(384)*((E*Ii)/li^4):
```

```
> omega_i:= 22.4*sqrt((E*Ii)/(rho*hi*li^4)):
> T_i:=(2*PI)/omega_i:
> ui1(t):=1000*((P0/ki)*(1-(t/td)-cos(omega_i*t)+(1/(omega_i*td))*sin(omega_i*t))):
> ui2(t):=1000*((P0/ki)*((sin(omega_i*t)/(omega_i*td))-(sin(omega_i*(t-td))/(omega_i*td))-
cos(omega_i*t))):
> ui(t):=piecewise(t<td,ui1(t),t>td,ui2(t)):
```

Plot

```
> plot([ur(t),uf(t),uo(t),ui(t)],t=0..0.1,legend=["Roof","Floor","Outerwall","Intermediate
wall"],labels=["t [s]","Deflection [mm]"],labeldirections=[horizontal,vertical]);
```

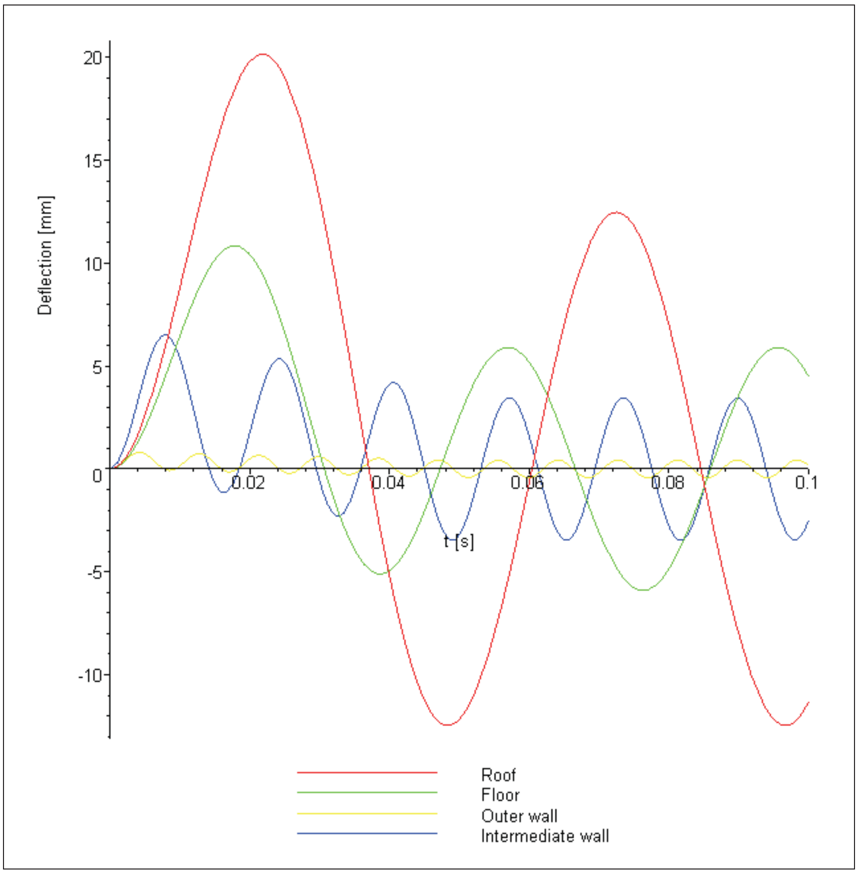


Figure A - 1 Output mass spring model

Appendix B Spread sheet vertical stability

In order to check the vertical stability of the cross-section that was developed by BAM Infraconsult for the second Coen tunnel, a spreadsheet is developed. The lay out of this spreadsheet is presented below, the calculation is made per meter length.

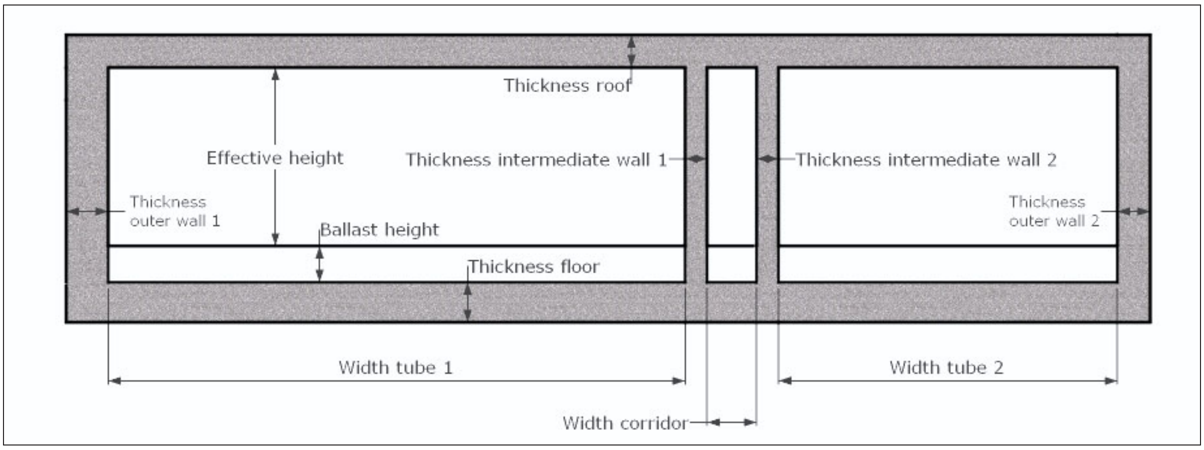


Figure B-1 Parameters to be varied

(light blue)	Chosen value	
(yellow)	Fixed or calculated value	

ρ reinforced concrete high	25.00	[kN/m^3]
ρ reinforced concrete low	24.00	[kN/m^3]
ρ non reinforced concrete	22.50	[kN/m^3]
ρ water transport low	10.00	[kN/m^3]
ρ water final high	10.35	[kN/m^3]
Thickness floor slab	1.10	[m]
Thickness roof slab	0.90	[m]
Thickness intermediate wal 1	0.60	[m]
Thickness intermediate wal 2	0.60	[m]
Thickness outer wall 1	1.15	[m]
Thickness outer wall 2	0.90	[m]
Width tube 1	15.75	[m]
Width tube 2	9.25	[m]
Width corridor	1.35	[m]
Inner height effective	4.60	[m]
balast heigth	1.30	[m]
Overall width	29.60	[m]
Overall height	7.90	[m]
Structural concrete	78.88	[m^2]
Balast concrete	34.26	[m^2]
Bulk heads + immersion equipement	30.00	[kN/m]
Water balast	0.00	[kN]
Volume	234.34	[m^3]
Maximal buoyancy force	2343.40	[kN]
Weight	2001.88	[kN]
Weight/ max buoyancy force	0.85	[-]
freeboard	1.15	[m]
Safety final situation	1.14	[-]

Table B-1 Paramters vertical stability

Apart from the freeboard and safety with respect to uplift in the final situation, a picture of the element during transport is automatically generated in this spreadsheet.

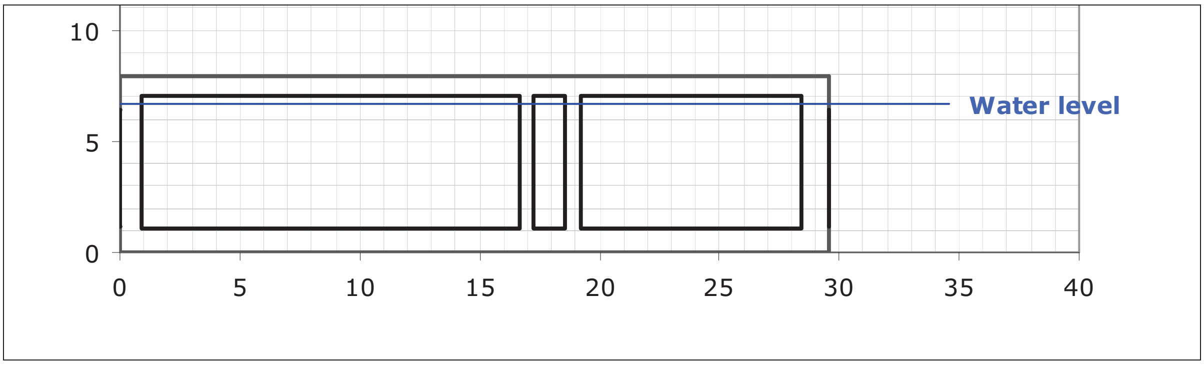


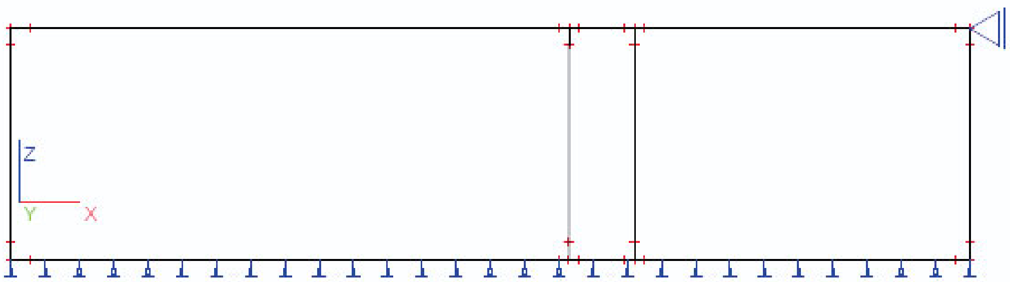
Figure B-2 Element during transport

Appendix C Results ESA PT without explosion

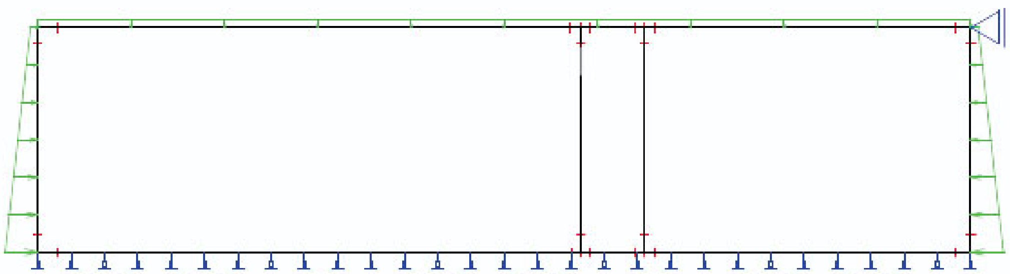
The adopted cross-section is modeled with the software package ESA PT and structural analysis is performed. The results of the calculation are presented below.

Project	Case study
Onderdeel	Cross-section
Omschrijving	Without explosion
Auteur	D.J. de Jong

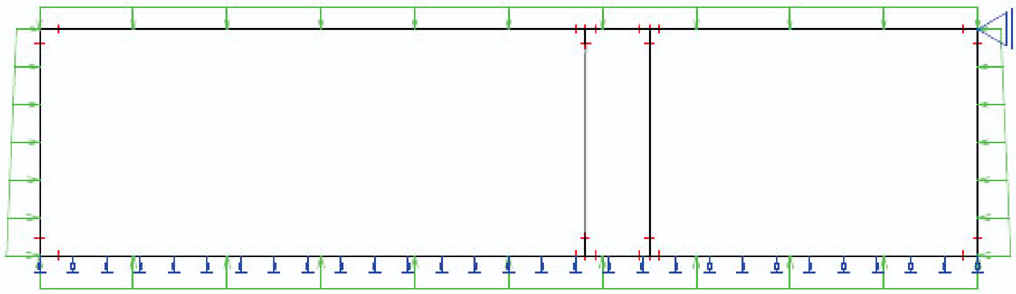
1. Structural scheme



2. Soil pressure

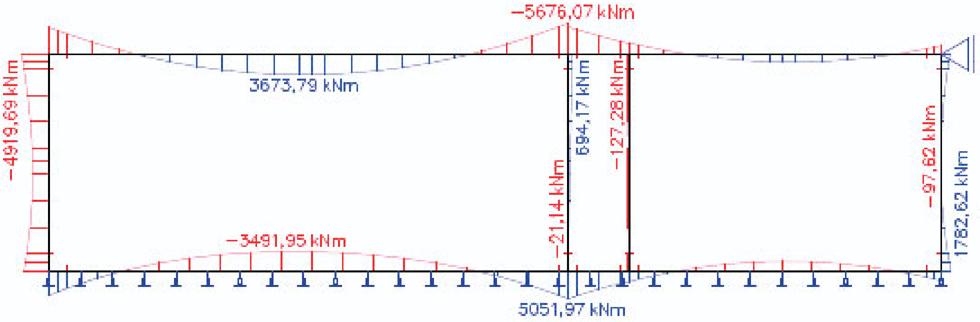


3. Water pressure

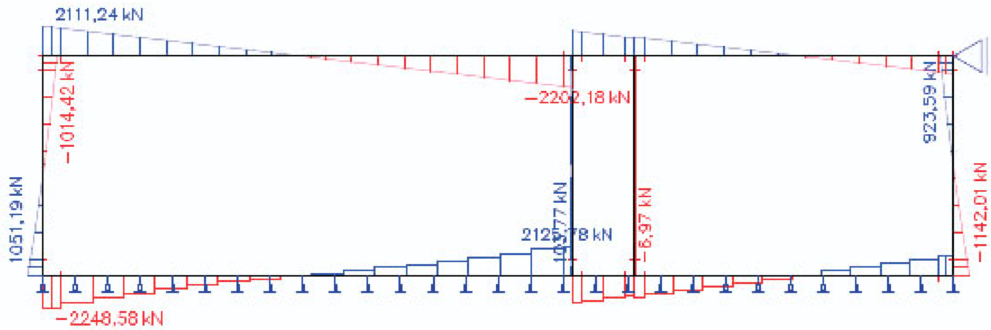


Project	Case study
Onderdeel	Cross-section
Omschrijving	Without explosion
Auteur	D.J. de Jong

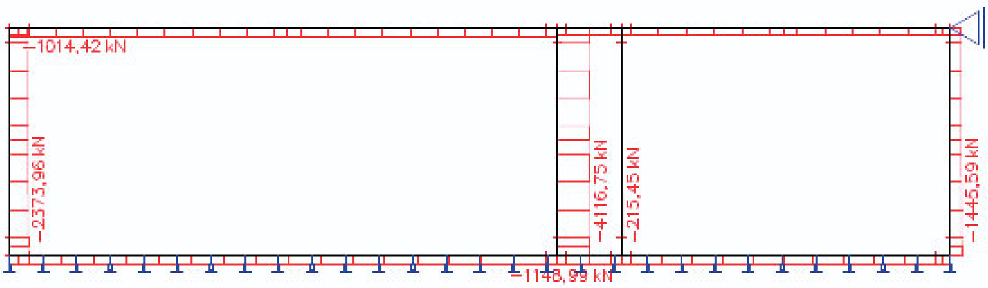
4. Bending moments ULS



5. Shear forces ULS

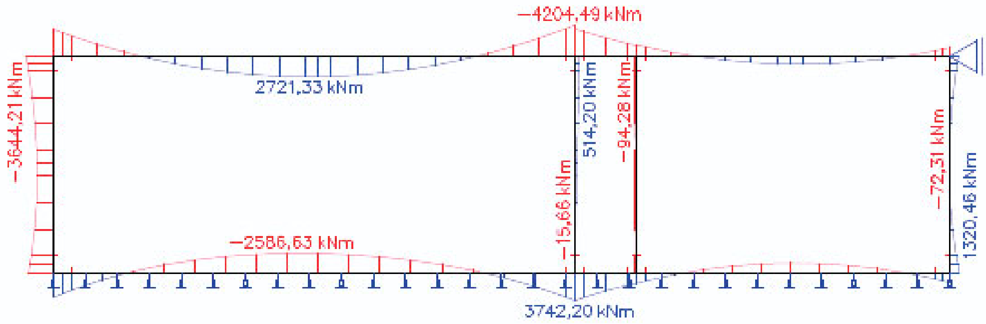


6. Normal forces ULS

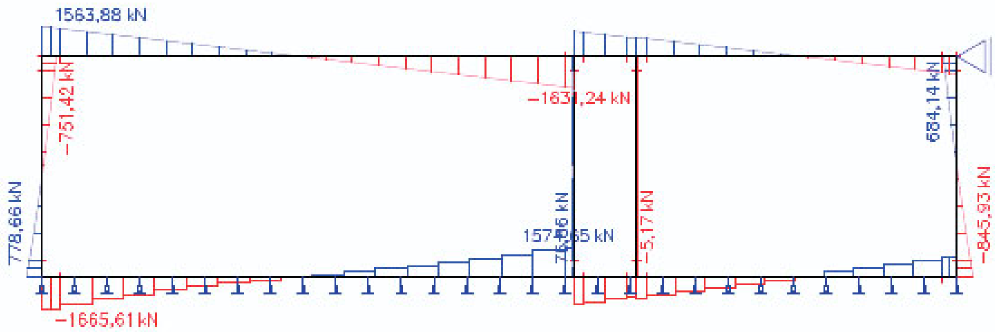


Project	Case study
Onderdeel	Cross-section
Omschrijving	Without explosion
Auteur	D.J. de Jong

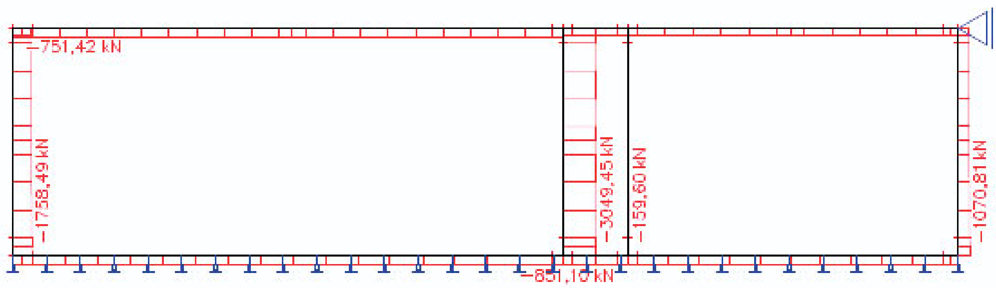
7. Bending moments SLS



8. Shear forces SLS

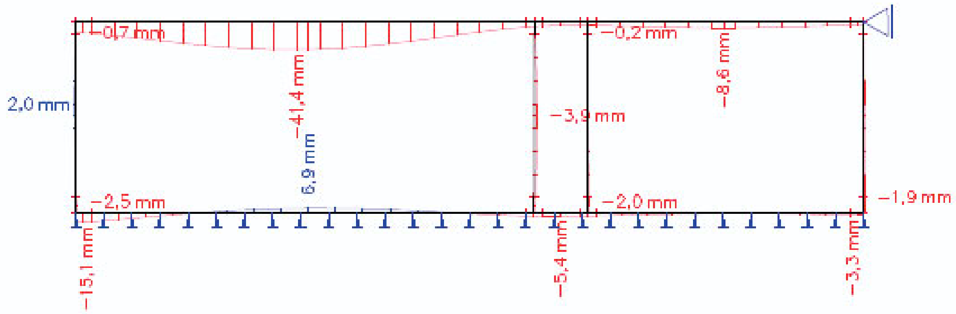


9. Normal forces SLS



Project	Case study
Onderdeel	Cross-section
Omschrijving	Without explosion
Auteur	D.J. de Jong

10. Deflections



Appendix D Spreadsheet without explosion

In order to determine the required amount of bending and shear force reinforcement, a spread sheet is developed. The lay out of this spread sheet is presented below. For each structural element of the cross-section similar calculations are made, for clearness only the calculations for the roof slab are presented in this appendix.

Design calculations cross-section without explosion

Key

	Chosen value
	Fixed or calculated value
	Result from calculation with ESA

Materials

Concrete

class	f'_{ck} [N/mm²]	$f_{bm,o}$ [N/mm²]	E'_b [N/mm²]	n [-]	ω_{min} [%]	ω_{max} [%]
B35	35	3.21	31000	6.45	0.18	1.93

cover [mm]	α	β
50	0.75	0.39

$g_{c;normal}$	1.4
α_{normal}	0.85
$\alpha\, f_{cd;normaal}$	21

Reinforcement

type	f_{srep} [N/mm²]	f_s [N/mm²]	E_s [N/mm²]	ε_{su} [%]
FeB 500	500	500	200000	3.25

γ_s	1.15

1. Roof next to support

Bending moment (ULS)

b	1000	[mm]
h	1000	[mm]

	ULS	SLS	
M_d	3749	2776	[kNm]
V_d	1962	1453	[kN]
N'_d	1014	751	[kN]

1	Ø 32	-	100	d_1	934	[mm]
2	Ø 32	-	150	d_2	852	[mm]
				$d_{average}$	893	[mm]

spacing between layers 50 [mm]

A_s	13404	[mm²]
ω	0.0150	
N_s	5828	[kN]
x_u	306	[mm]
z	774	[mm]

$$\omega = \frac{A_s}{b \cdot d}$$

$$N_s = A_s \cdot f_s$$

$$x_u = \frac{N_s - N'_d}{b \cdot \alpha f_{cd;normal}}$$

$$z = d_{average} - \beta x_u$$

M_u	4896	>	3749	[kNm]	ok
$A_{s;min} = \omega_{min} \cdot b \cdot d$	1607	<	13404	[mm²/m]	ok
$A_{s;max} = \omega_{max} \cdot b \cdot d$	17235	>	13404	[mm²/m]	ok

Shear force (ULS)

b	1000	[mm]
h	900	[mm]

Ø 20 - 100 - 900 [mm]

Asw	698	[mm²]
v	0.516	[-]
σ_{cw}	1	[-]
v_{rd}	2349	[kN]
v_{rdmax}	4192	[kN]

$$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right]$$

Euro code

$$V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot (f_s / \gamma_s) \cot\theta$$

$$V_{rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu \cdot f_{cd} / (\cot\theta + \tan\theta)$$

v_{rd}	2349	>	1962[kN]	ok

Crack width (SLS)

w_{max}	0.3[mm]			
σ_s	258[N/mm²]			
$Ø_{max}$	12	>	32 [mm]	not ok
s_{max}	150	≥	150 [mm]	ok
Crackwidth requirement				ok

2 Roof mid span

Bending moment (ULS)

b	1000	[mm]
h	900	[mm]

	ULS	SLS
M_d	4300	2721[kNm]
V_d	0	0[kN]
N'_d	1014	751[kN]

1	Ø 32	-	100	d_1	884	[mm]
2	Ø 32	-	150	d_2	802	[mm]
				$d_{average}$	843	[mm]
spacing between layers			50	[mm]		

A_s	13404[mm²]
ω	0.0159
N_s	5828[kN]
x_u	306[mm]
z	724[mm]

$$\omega = \frac{A_s}{b \cdot d}$$
$$N_s = A_s \cdot f_s$$
$$x_u = \frac{N_s + N_d'}{b \cdot \alpha f_{cd;normal}}$$
$$z = d_{average} - \beta x_u$$

M_u	4614	>	4300[kNm]	ok
$A_{s;min} = \omega_{min} \cdot b \cdot d$	1517	<	13404[mm²/m]	ok
$A_{s;max} = \omega_{max} \cdot b \cdot d$	16270	>	13404[mm²/m]	ok

Shear force (ULS)

b	1000	[mm]
h	900	[mm]

Ø 16	-	200	-	900	[mm]

Asw	447	[mm²]	
v	0.516	[-]	$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right]$
σ_{cw}	1	[-]	Euro code
V_{rd}	703	[kN]	$V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot (f_s / \gamma_s) \cot\theta$
V_{rdmax}	3922	[kN]	$V_{rd,\max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu \cdot f_{cd} / (\cot\theta + \tan\theta)$

V_{rd}	703	>	0	[kN]	ok

Crack width (SLS)

w_{max}	0.3	[mm]			
σ_s	268	[N/mm²]			
$Ø_{max}$	12	>	32	[mm]	not ok
s_{max}	150	≥	150	[mm]	ok
Crackwidth requirement					ok

3 Roof next to support

Bending moment (ULS)

b	1000	[mm]
h	1000	[mm]

	ULS	SLS	
M_d	4995	3700	[kNm]
V_d	2120	1570	[kN]
N'_d	1014	751	[kN]

1	Ø 32	-	100	d_1	984	[mm]
2	Ø 32	-	100	d_2	902	[mm]
				$d_{average}$	943	[mm]

spacing between layers 50 [mm]

A_s	16085	[mm²]

ω	0.0171	
N_s	6993	[kN]
x_u	380	[mm]
z	795	[mm]

M_u	5991	>	4995	[kNm]	ok
$A_{s;min} = \omega_{min} \cdot b \cdot d$	1697	<	16085	[mm²/m]	ok
$A_{s;max} = \omega_{max} \cdot b \cdot d$	18200	>	16085	[mm²/m]	ok

Shear force (ULS)

b	1000	[mm]
h	1000	[mm]

$$\omega = \frac{A_s}{b \cdot d}$$

Ø 20	-	100	-	900		[mm]

Asw	698	[mm²]

$$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right]$$

v	0.516	[-]
σ_{cw}	1	[-]

Euro code

$$V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot (f_s / \gamma_s) \cot\theta$$

V_{rd}	2413	[kN]
V_{rdmax}	4307	[kN]

$$V_{rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu \cdot f_{cd} / (\cot\theta + \tan\theta)$$

V_{rd}	2413	>	2120	[kN]	ok

Crack width (SLS)

w_{max}	0.3	[mm]			
σ_s	271	[N/mm²]			
$Ø_{max}$	12	>	32	[mm]	not ok
s_{max}	150	≥	100	[mm]	ok
Crackwidth requirement					ok

Appendix E Required reinforcement without explosion

For the determination of the reinforcement, the following locations are distinguished.

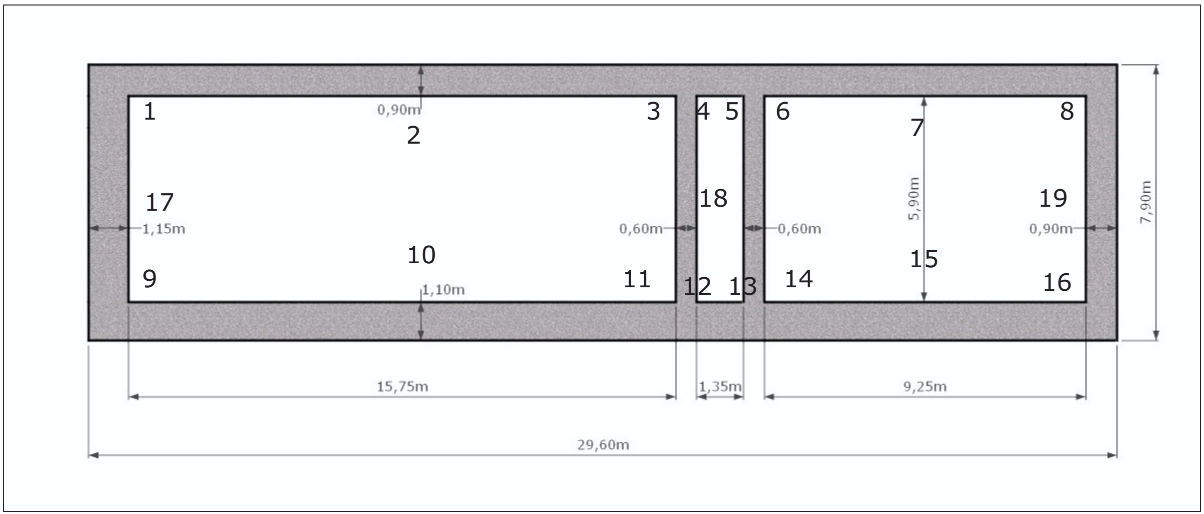


Figure E-1 Locations considered for the determination of the reinforcement

The required amount of bending and shear force reinforcement is determined according to the Eurocode. The results are listed in table

Element	Location	Longitudinal	Location	Stirrups	
Roof	1	Ø 32 - 100 mm	top	Ø 20 - 100 - 900 mm	178 kg/m³
		Ø 32 - 150 mm	top		
Roof	2	Ø 32 - 100 mm	bottom	Ø 16 - 200 - 900 mm	136 kg/m³
		Ø 32 - 150 mm	bottom		
Roof	3	Ø 32 - 100 mm	top	Ø 20 - 100 - 900 mm	201 kg/m³
		Ø 32 - 100 mm	top		
Roof	4	Ø 32 - 100 mm	top	Ø 20 - 100 - 900 mm	201 kg/m³
		Ø 32 - 100 mm	top		
Roof	5	Ø 32 - 150 mm	top	Ø 20 - 150 - 900 mm	134 kg/m³
		Ø 32 - 150 mm	top		
Roof	6	Ø 32 - 200 mm	top	Ø 20 - 150 - 900 mm	111 kg/m³
		Ø 32 - 200 mm	top		
Roof	7	Ø 32 - 200 mm	bottom	Ø 16 - 200 - 900 mm	68 kg/m³
		Ø 20 - 200 mm	bottom		
Roof	8	Ø 32 - 200 mm	top	Ø 20 - 150 - 900 mm	104 kg/m³
		Ø 32 - 250 mm	top		
Floor	9	Ø 32 - 150 mm	top	Ø 20 - 100 - 900 mm	126 kg/m³
		Ø 32 - 150 mm	top		
Floor	10	Ø 32 - 150 mm	bottom	Ø 16 - 200 - 900 mm	92 kg/m³
		Ø 32 - 150 mm	bottom		
Floor	11	Ø 32 - 100 mm	top	Ø 20 - 100 - 900 mm	145 kg/m³
		Ø 32 - 150 mm	top		
Floor	12	Ø 32 - 100 mm	top	Ø 20 - 150 - 900 mm	129 kg/m³
		Ø 32 - 150 mm	top		
Floor	13	Ø 32 - 200 mm	top	Ø 20 - 150 - 900 mm	91 kg/m³
		Ø 32 - 200 mm	top		
Floor	14	Ø 32 - 250 mm	top	Ø 20 - 200 - 900 mm	71 kg/m³
		Ø 32 - 250 mm	top		
Floor	15	Ø 32 - 200 mm	bottom	Ø 16 - 200 - 900 mm	73 kg/m³
		Ø 32 - 200 mm	bottom		
Floor	16	Ø 20 - 200 mm	top	Ø 16 - 200 - 900 mm	56 kg/m³
		Ø 32 - 200 mm	top		
Outer wall	1	Ø 32 - 100 mm	top	Ø 16 - 200 - 900 mm	110 kg/m³
		Ø 32 - 150 mm	top		
Outer wall	9	Ø 32 - 100 mm	top	Ø 16 - 200 - 900 mm	110 kg/m³
		Ø 32 - 150 mm	top		
Outer wall	16	Ø 32 - 250 mm	top	Ø 20 - 200 - 900 mm	87 kg/m³
		Ø 32 - 250 mm	top		
Outer wall	8	Ø 32 - 250 mm	top	Ø 16 - 150 - 900 mm	82 kg/m³
		Ø 32 - 250 mm	top		
Outer wall	17	Ø 32 - 150 mm	top	Ø 16 - 200 - 900 mm	91 kg/m³
		Ø 32 - 150 mm	top		
Outer wall	19	Ø 16 - 200 mm	top	Ø 16 - 200 - 900 mm	37 kg/m³
		Ø 16 - 200 mm	top		
Intermediate wall	11	Ø 16 - 200 mm	top	Ø 16 - 200 - 900 mm	42 kg/m³
		Ø - 150 mm	top		
Intermediate wall	3	Ø 32 - 150 mm	top	Ø 16 - 200 - 900 mm	120 kg/m³
		Ø 20 - 200 mm	top		
Intermediate wall	14	Ø 32 - 200 mm	top	Ø 20 - 150 - 900 mm	130 kg/m³
		Ø 20 - 250 mm	top		
Intermediate wall	6	Ø 20 - 250 mm	top	Ø 16 - 200 - 900 mm	62 kg/m³
		Ø 20 - 250 mm	top		
Intermediate wall	18	Ø 16 - 150 mm	top	Ø 16 - 200 - 900 mm	60 kg/m³
		Ø 16 - 200 mm	top		

Table E - 1 required amounts of reinforcement

The location of the determined bending reinforcement is indicated in the figure below, the diagram of bending moments is also depicted.

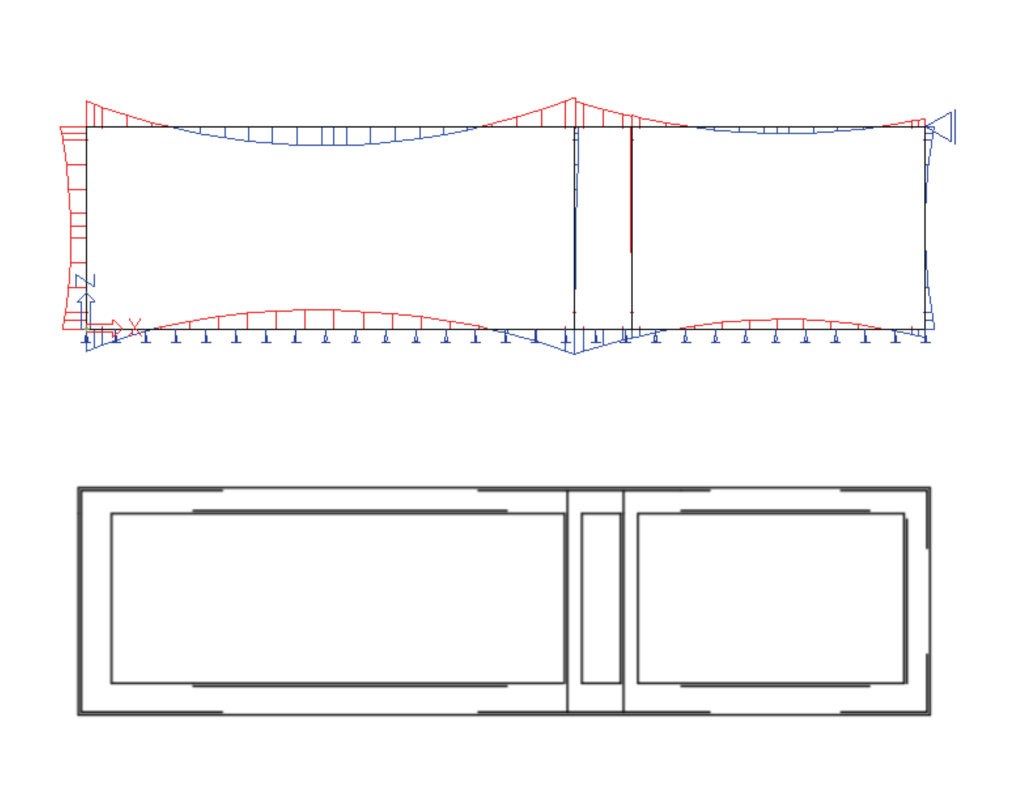


Figure E-2 Envelope diagram bending moments and locations for reinforcement

From the bending moment diagram, the length at which the reinforcement should be applied can be estimated. Estimations for the total required amount of reinforcement can be made subsequently. Since for a number of locations the differences are only small, the following areas will be distinguished.

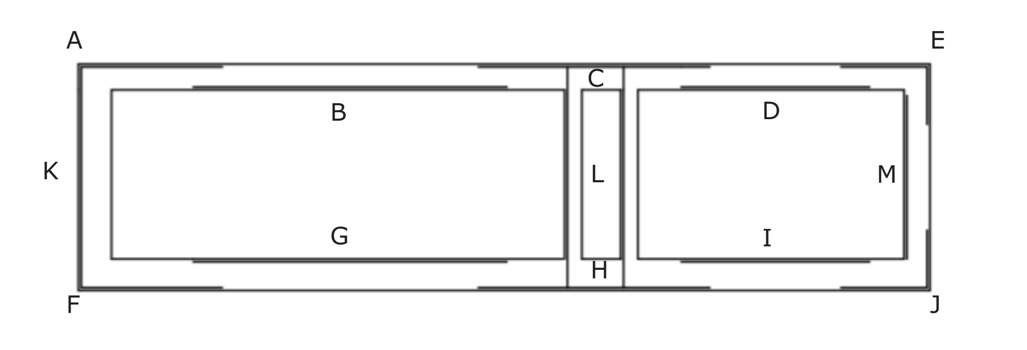


Figure E-3 Different areas for the reinforcement

Location	Element	Length	Reinforcement [kg/m]	Concrete [m²/m]	[kg/m³]	Reinforcement [kg]
A	Roof	5	160	0.9	178	800
B	Roof	10	123	0.9	136	1228
C	Roof	8	181	0.9	201	1449
D	Roof	7	61	0.9	68	430
E	Roof	3	93	0.9	104	280
F	Floor	5	139	1.1	126	695
G	Floor	10	102	1.1	92	1017
H	Floor	8	142	1.1	129	1134
I	Floor	7	81	1.1	73	565
J	Floor	3	61	1.1	56	184
A	Outer wall 1	2	123	1.115	110	246
K	Outer wall 1	5	102	1.115	91	509
F	Outer wall 1	2	123	1.115	110	246
C	Intermediate wall 1	2	72	0.6	120	144
L	Intermediate wall 1	5	36	0.6	60	180
H	Intermediate wall 1	2	25	0.6	42	51
C	Intermediate wall 2	2	72	0.6	120	144
L	Intermediate wall 2	5	36	0.6	60	180
H	Intermediate wall 2	2	25	0.6	42	51
E	Outer wall 2	2	74	0.9	82	148
M	Outer wall 2	5	33	0.9	37	167
J	Outer wall 2	2	78	0.9	87	156

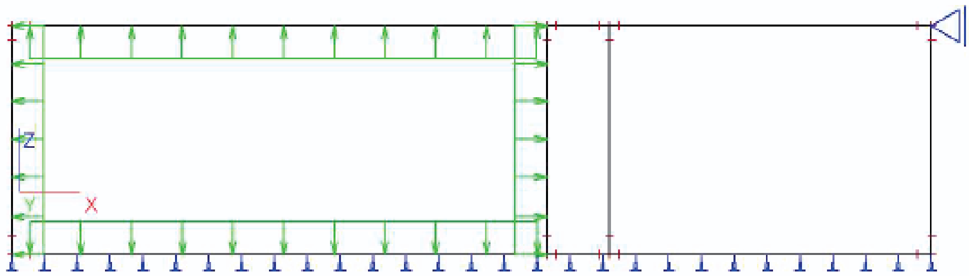
Table E-2 Estimation for the required amount of reinforcement

Appendix F Output ESA with explosion load

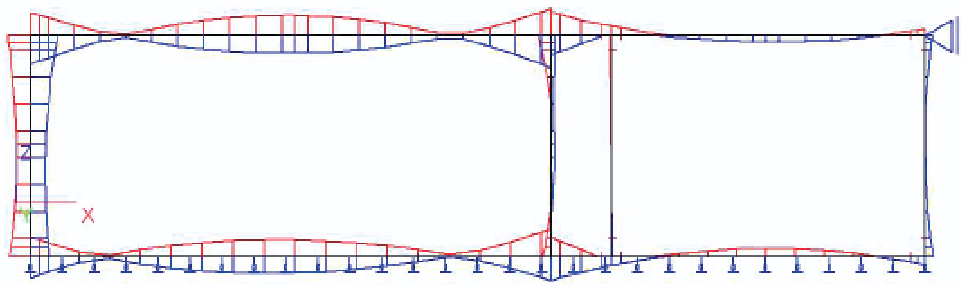
The following figures illustrate the envelope diagrams of the forces that occur as a result of a statically outward directed explosion load with a magnitude of 500 kPa.

Project	Case study
Onderdeel	Cross-section
Omschrijving	With explosion load
Auteur	D.J. de Jong

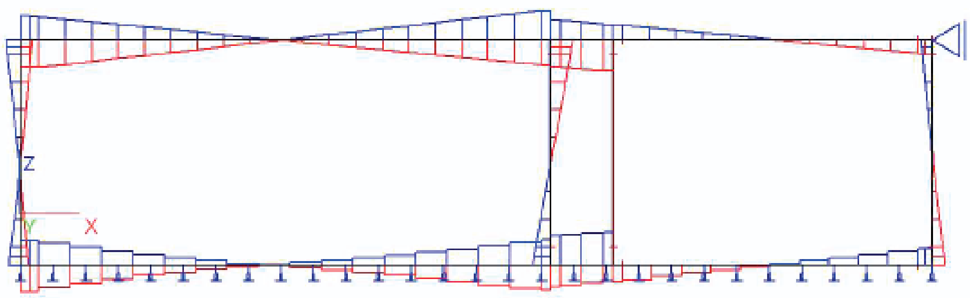
1. Explosion load large tube



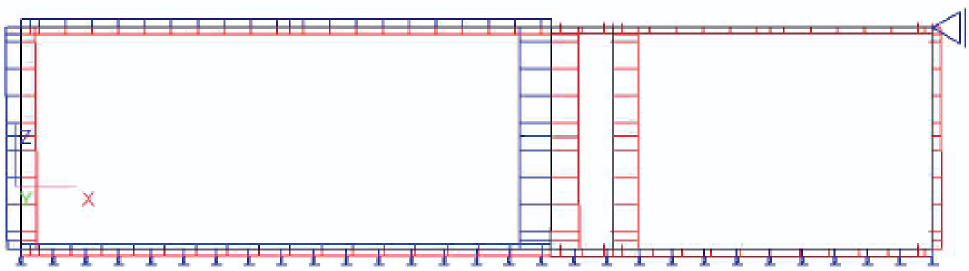
2. Envelope diagram bending moments



3. Envelope diagram shearforces



4. Envelope diagram axial forces



Appendix G Spread sheet with explosion

For the calculation, taking the explosion load into account, some adaptations to the spreadsheet are made. The lay out is presented below. This example concerns the calculation for the roof. The remaining calculations are not attached, though these are available if desired.

Design calculations cross-section with explosion

Key

(blue)	Chosen value
(yellow)	Fixed or calculated value
(orange)	Result from calculation with ESA

Materials

Concrete

class	f'_{ck} [N/mm²]	$f_{bm,o}$ [N/mm²]	E'_b [N/mm²]	n [-]	ω_{min} [%]	ω_{max} [%]
B35	35	3.21	31000	6.45	0.18	4

cover [mm]	α	β
50	0.75	0.39

$g_{c;normal}$	1.4
a_{normal}	0.85
$a\ f_{cd;normaal}$	21

Reinforcement

type	f_{srep} [N/mm²]	f_s [N/mm²]	E_s [N/mm²]	ε_{su} [%]
FeB 500	500	500	200000	3.25

γ_s	1.15

1. Roof next to support outer wall

Bending moment (ULS)

b	1000	[mm]
h	1100	[mm]

	SLS + explosion outward	SLS + explosion inward
M_d	1641	-6100
V_d	-770	2700
N'_d	-470	1493

outward
Bottom reinforcement

1	Ø 20	-	100	d_1	1040	[mm]
2	Ø 20	-	200	d_2	970	[mm]
				$d_{average}$	1005	[mm]

spacing between layers 50 [mm]

A_s 4712 [mm²]

N_s	2049[kN]	$N_s = A_s \cdot f_s$
x_u	91[mm]	$x_u = \frac{N_s + N_d'}{b \cdot \alpha f_{cd;normal}}$
z	969[mm]	$z = d_{average} - \beta x_u$

Inward

Top reinforcement

1 Ø 40	-	150	d_1	1030	[mm]
2 Ø 40	-	120	d_2	940	[mm]
			$d_{average}$	985	[mm]

spacing between layers 50 [mm]

A_s	18850[mm²]	
N_s	8195[kN]	$N_s = A_s \cdot f_s$
x_u	615[mm]	$x_u = \frac{N_s + N_d'}{b \cdot \alpha f_{cd;normal}}$
z	745[mm]	$z = d_{average} - \beta x_u$

$M_{u\ explosion\ outward}$	1744	>	1641[kNm]	**ok**
$M_{u\ explosion\ inward}$	6569	>	6100[kNm]	**ok**
$A_{s;\max} = \omega_{\max} \cdot b \cdot h$	40200	>	21420[mm²/m]	**ok**

Shear force

b	1000 [mm]
h	900 [mm]

Ø 32 - 200 - 900

Asw	1787[mm²]	
v	0.516[-]	$v = 0.6\left[1 - \frac{f_{ck}}{250}\right]$
σ_{cw}	1[-]	Euro code
V_{rd}	2895[kN]	$V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot (f_s / \gamma_s) \cot\theta$
V_{rdmax}	4037 [kN]	$V_{rd,\max} = \alpha_{cw} \cdot b_w \cdot z \cdot v \cdot f_{cd} / (\cot\theta + \tan\theta)$
V_{rd}	2895 > 2700[kN]	**ok**

2. Roof mid span

Bending moment (ULS)

b	1000	[mm]
h	900	[mm]

	SLS + explosion outward	SLS + explosion inward
M_d	-1480	5200
V_d	100	250
N'_d	-470	1493

outward

Top reinforcement

1 Ø 20	-	100	d_1	840	[mm]
2 Ø 20	-	150	d_2	770	[mm]
			$d_{average}$	805	[mm]
spacing between layers		50	[mm]		

A_s	5236	[mm²]
N_s	2277	[kN]
x_u	127	[mm]
z	755	[mm]

$$N_s = A_s \cdot f_s$$

$$x_u = \frac{N_s + N_d'}{b \cdot \alpha f_{cd;normal}}$$

$$z = d_{average} - \beta x_u$$

Inward

Bottom reinforcement

1 Ø 40	-	100	d_1	830	[mm]
2 Ø 32	-	80	d_2	744	[mm]
			$d_{average}$	787	[mm]
spacing between layers		50	[mm]		

A_s	22619	[mm²]
N_s	9835	[kN]
x_u	719	[mm]
z	507	[mm]

$$N_s = A_s \cdot f_s$$

$$x_u = \frac{N_s + N_d'}{b \cdot \alpha f_{cd;normal}}$$

$$z = d_{average} - \beta x_u$$

$M_{u\ explosion\ outward}$	1531	>	1480	[kNm]	**ok**
$M_{u\ explosion\ inward}$	5234	>	5200	[kNm]	**ok**
$A_{s;max} = \omega_{max} \cdot b \cdot h$	32200	>	30951	[mm²/m]	**ok**

Shear force

b	1000	[mm]
h	900	[mm]

Ø 16	-	200	-	900

Asw	447	[mm²]	
v	0.516	[-]	$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right]$
σ_{cw}	1	[-]	Euro code
v_{rd}	492	[kN]	$V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot (f_s / \gamma_s) \cot\theta$
v_{rdmax}	2744	[kN]	$V_{rd,\max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu \cdot f_{cd} / (\cot\theta + \tan\theta)$
v_{rd}	492	>	250 [kN] ok

3. Roof next to support intermediate wall

Bending moment (ULS)

b	1000	[mm]
h	1100	[mm]

	SLS + explosion outward	SLS + explosion inward
M_d	2100	-7150
V_d	730	2700
N'_d	-470	1493

outward
Bottom reinforcement

1	Ø 32	-	150	d_1	1034	[mm]
2	Ø 20	-	100	d_2	958	[mm]
				$d_{average}$	996	[mm]

spacing between layers 50 [mm]

A_s	8503	[mm²]
N_s	3697	[kN]
x_u	186	[mm]
z	923	[mm]

$$N_s = A_s \cdot f_s$$

$$x_u = \frac{N_s + N_d'}{b \cdot \alpha f_{cd;normal}}$$

$$z = d_{average} - \beta x_u$$

Inward
Top reinforcement

1	Ø 40	-	100	d_1	1030	[mm]
2	Ø 40	-	100	d_2	940	[mm]
				$d_{average}$	985	[mm]

spacing between layers 50 [mm]

A_s	25133	[mm²]
N_s	10927	[kN]

$$N_s = A_s \cdot f_s$$

x_u	789	[mm]			
z	677	[mm]			

$$z = d_{average} - \beta x_u$$

$$x_u = \frac{N_s + N_d}{b \cdot \alpha f_{cd;normal}}$$

$M_{u\ explosion\ outward}$	3189	>	2100	[kNm]	ok
$M_{u\ explosion\ inward}$	7765	>	7150	[kNm]	ok
$A_{s;max} = \omega_{max} \cdot b \cdot h$	39840	>	30578	[mm²/m]	ok

Shear force

b	1000	[mm]
h	900	[mm]

Ø 32	-	150	-	900

Asw	1787	[mm²]
v	0.516	[-]
σ_{cw}	1	[-]
v_{rd}	3509	[kN]
v_{rdmax}	3670	[kN]

$$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right]$$

Euro code

$$V_{rd,s} = \frac{A_{sw}}{s} \cdot z \cdot (f_s / \gamma_s)\cot\theta$$

$$V_{rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu \cdot f_{cd} / (\cot\theta + \tan\theta)$$

v_{rd}	3509	>	2700	[kN]	ok

Appendix H Design with explosion load

H.1 Design for scenario 1

It will be checked if it is possible to design the initially assumed cross-section for scenario 1. To this end, similar calculations as performed for the scenario in which no explosion load is taken into account will be made. The estimated cross-section is modelled in ESA PT. The occurring bending moments, shear forces and axial forces are calculated. Subsequently, the required bending and shear reinforcement is determined according to the Euro code. An extensive report concerning the calculation performed with ESA PT, as well as the spreadsheets that are used for the determination of the reinforcement is attached in Appendix E. The results of this calculation are summarized in the table below. Since the spans of the two tubes differ considerably, distinction is made.

Location	Element	Length [m]	Reinforcement [kg/ m]	[Concrete m²/m]	[kg/m³]	Reinforcement [kg]
A	Roof tube 1	5	255	1.1	232	1276
B	Roof tube 1	10	236	0.9	262	2362
C	Roof tube 1	4	358	1.1	325	1430
F	Floor tube 1	5	251	1.1	228	1255
G	Floor tube 1	10	150	1.1	136	1497
H	Floor tube 1	4	289	1.1	262	1154
A	Outer wall 1	2	202	1.115	182	405
K	Outer wall 1	5	113	1.115	101	565
F	Outer wall 1	2	161	1.115	145	323
C	Intermediate wall 1	2	171	0.7	245	343
L	Intermediate wall 1	5	62	0.6	104	311
H	Intermediate wall 1	2	129	0.7	184	258
C	Roof tube 2	3	140	0.9	155	419
D	Roof tube 2	7	79	0.9	88	552
E	Roof tube 2	3	206	0.9	229	617
H	Floor tube 2	3	95	1.1	86	284
I	Floor tube 2	7	77	1.1	70	541
J	Floor tube 2	3	81	1.1	74	243
C	Intermediate wall 2	2	118	0.6	196	235
L	Intermediate wall 2	5	83	0.6	138	413
H	Intermediate wall 2	2	106	0.6	177	212
E	Outer wall 2	2	86	0.9	96	173
M	Outer wall 2	5	52	0.9	57	259
J	Outer wall 2	2	86	0.9	96	172

Table H-1 Required amount of reinforcement with scenario 1

Concrete	79	[m³]
Reinforcement	15298	[kg/m]
Overall ratio	194	[kg/m³]

Table H-2 Quantities per meter

It can be concluded that it is possible to design the assumed cross-section for the explosion load. At the connections with the intermediate walls, the thickness of the roof slab should be increased slightly. This measure is required for a very limited length and is common for tunnels. The thickness of the roof is usually enlarged near the supports and inclines towards a constant value within a distance of 1 meter approximately. The additional concrete volume due to this measure is only limited and is neglected for this rough evaluation.

For tube 1 very high amounts of reinforcement are required, which is unfavourable for the execution phase. For tube 2 the magnitude of the required reinforcement is considerably lower.

Interaction between elements

Since all elements are loaded, the consequence of the explosion is reinforce3d by a secondary effect. The load on an element namely causes a tensile force in elements that are situated perpendicular to it. The pressure that acts on the roof and floor results in a tensile force in the intermediate wall for instance. Therefore the capacity against bending moments of the intermediate wall decreases. Simultaneously the intermediate wall is loaded by the explosion pressure. Therefore the consequence of the explosion is reinforced, it worse than just the pressure acing on the considered element.

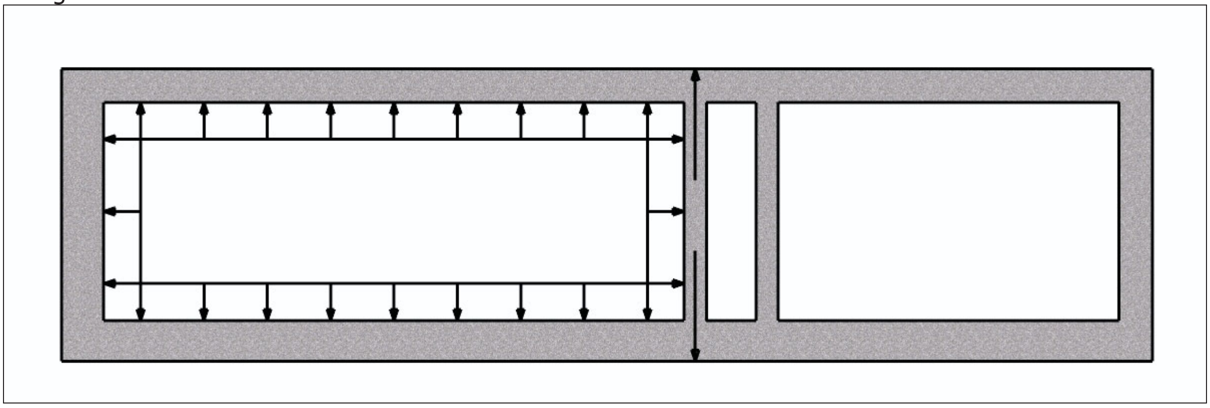


Figure H-1 Interaction between elements

For the inward directed part of the explosion the opposite holds, the capacity of the elements will be increased as a result of compression force that can be considered a sort of pre tense.

H.2 Design for scenario 2

From calculations it became clear that it is impossible to comply with the requirements concerning the explosion load for this scenario within the assumed cross-section. The maximum allowed percentages of reinforcement would be exceeded. Apart from that large diameters, up to Ø40 are required. Furthermore the amount of reinforcement per cubic meter of concrete would be excessively. Therefore it is inevitable to make adaptations to the cross-section.

Adaptations cross-section

Since the scenario considers a load with very large magnitude, the initially assumed cross-section will not comply with the requirements. Therefore adaptations are required.

- The cross-section should comply with the requirements for transport
- The effective space of the cross-section will not be decreased
- The dimensions of the cross-section will be adapted to provide sufficient capacity

In figure H-2 all parameters that can be varied in this respect are indicated.

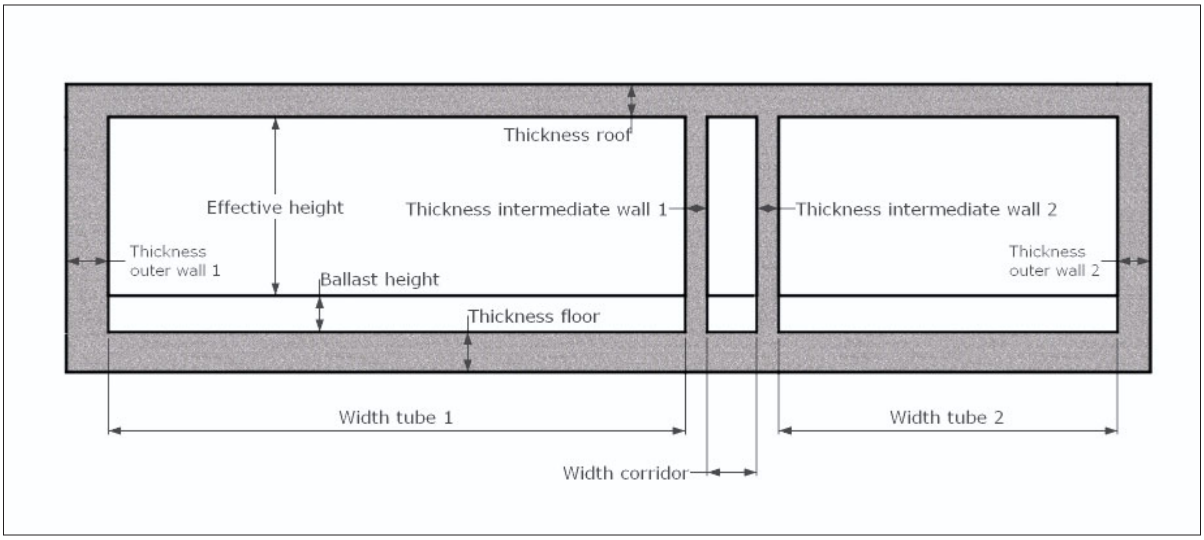


Figure H-2 Parameters to be varied

Since the magnitude of the explosion is large, the dimensions of the several elements should be increased. The following problem may arise however. Increasing the thicknesses slightly, results in a significantly increased deadweight of the element which makes it hard to comply with the requirement that the elements should be transported afloat, with the required freeboard. If this requirement is no longer met, the only option is to increase the hollow space in order to create extra buoyancy. This can be only achieved by increasing the length or height of the element, which will lead to greater spans. Due to larger spans, the occurring bending moments will be of greater magnitude. Additional reinforcement and possibly larger thicknesses will be required. Besides this, increasing the height of the elements is undesired because of the increased length of the tunnel, as explained in paragraph 2.1.6. The stated problem is schematically presented in the figure below.

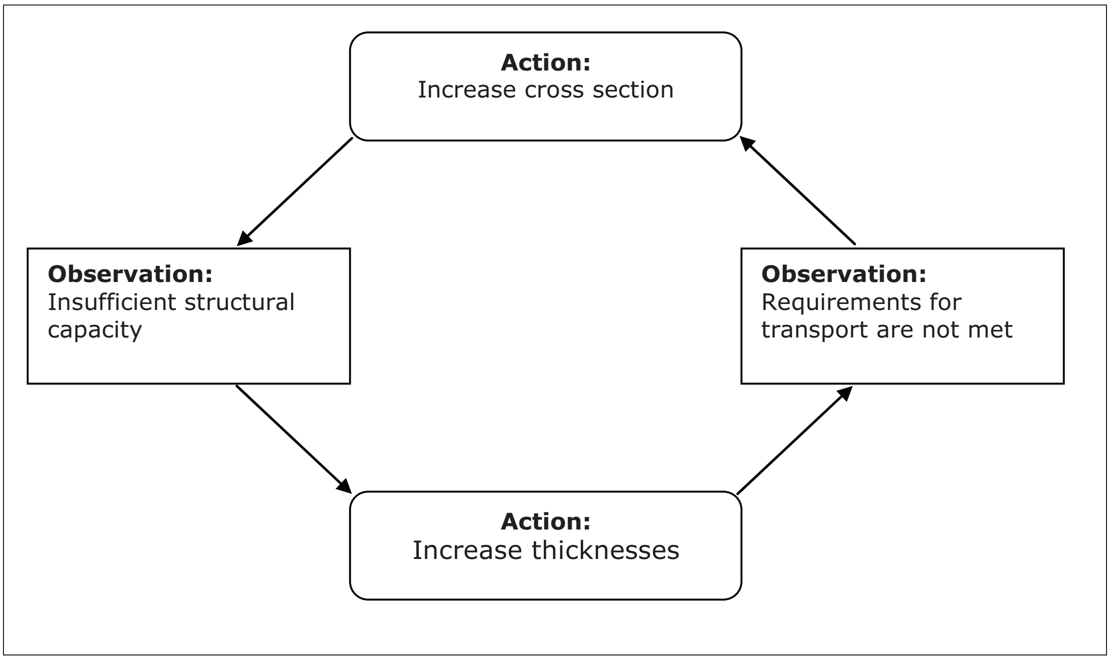


Figure H-3 Design problem

It is not desirable to change the effective height or widths of the cross-section. Any adaptation in fact results in inefficient space and use of materials. Adjusting these parameters is however the only possibility to comply with the requirements for transport. Therefore it is important to consider the drawbacks of the enlargement for each parameter in more detail. Subsequently it can be decided what is the best option.

- Since tube 1 has the largest span, it should be avoided to enlarge the inner width of this tube. Doing so, would result in bending moments of a larger magnitude and make the problem even worse.

- Increasing the width of the second tube is a possibility. Although no extra space is required, from a functional point of view. It could be considered to enlarge the capacity of the tube however.

- The width of the corridor can be increased, since the span between the intermediate walls is only limited, this will not result in any difficulties of structural nature. The space that is create is however useless.

- Increasing the height has a favourable influence to the buoyancy of the element, since it effects the entire width of the element. For the final stability, it may be necessary to apply additional ballast. An important drawback of increasing the height is that the tunnel as a whole will be situated deeper. This will result in additional dredging. Apart from that, the entrances needs to be longer, as a result of requirements concerning maximal curvature in vertical direction.

- Acquiring additional volume can be achieved by increasing the height and or width of the element. The element is approximately 30 m wide and 8 meters high. Therefore increasing the height is more efficient with respect to the amount of required building materials.

- From foregoing considerations it is decided to increase the height somewhat since otherwise a very wide element would be necessary. The thicknesses and spans of the elements are estimated and listed in the table below.

Parameter	Initially assumed cross-section (without explosion)	Estimated cross-section (scenario 2)
Thickness roof	0.9 m	1.4 m
Thickness floor	1.1 m	1.2 m
Thickness outer wall 1	1.15 m	1.2 m
Thickness outer wall 2	0.9 m	1.2 m
Thickness intermediate wall 1	0.6 m	0.8 m
Thickness intermediate wall 2	0.6 m	0.8 m
Width tube 1	15.75 m	15.75 m
Width tube 2	9.25 m	10 m
Width corridor	1.35 m	1.5 m
Effective height	4.9 m	5.7 m
Ballast height	1 m	1.3 m

Table H-3 Adaptations cross-section

The estimated cross-section complies with the requirement that the elements should be transported afloat. The calculation is done by means of a spreadsheet, similar to the one attached in Appendix B. The figure below is generated with this spreadsheet and presents the element during transport, the freeboard is 0.74 m.

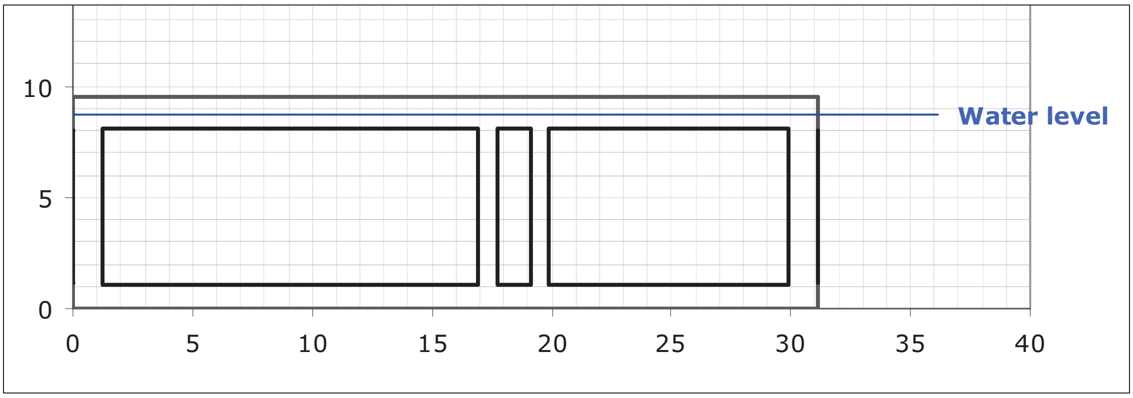


Figure H-4 Adapted cross-section during transport

It should be noted that the adapted cross-section is probably not the optimal solution, though for investigation of the effect of the load to a regular structure it is considered to be suitable.

Results preliminary structural design

In the figure below, the envelope diagram of the bending moments is presented as well as an indication for the location of the bending reinforcement.

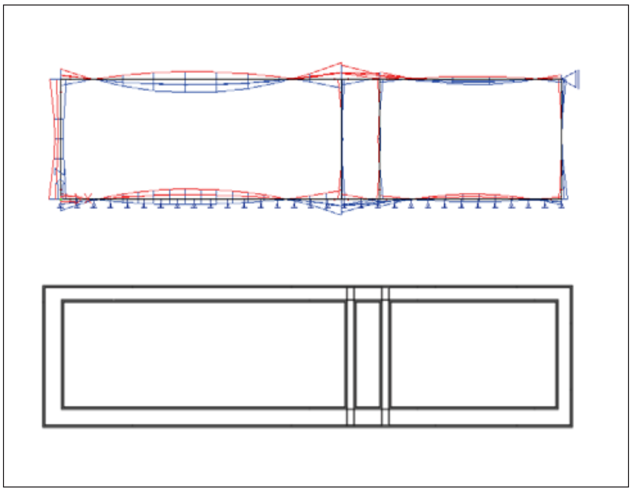


Figure H-5 Envelope diagram for the bending moments and location for the reinforcement

Since the structure can be loaded in two directions, significantly more bending reinforcement will be required. The results of the calculations are presented in the table below.

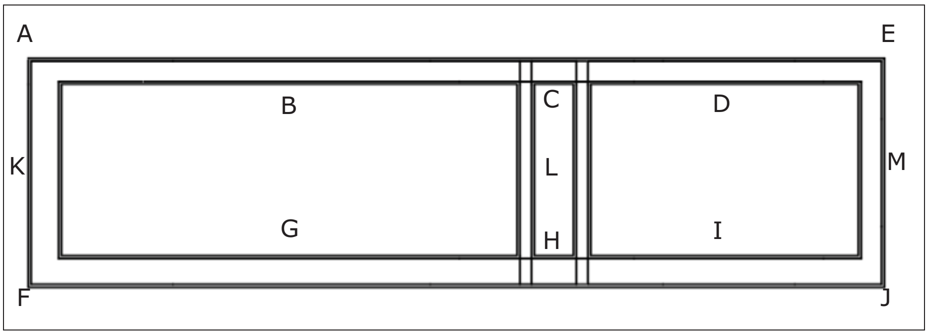


Figure H-7 Distinguished areas for the reinforcement

Location	Element	Length [m]	Reinforcement [kg/ m]	[Concrete m²/m]	[kg/m³]	Reinforcement [kg]
A	Roof tube 1	5	250	1.4	179	1251
B	Roof tube 1	10	247	1.4	176	2471
C	Roof tube 1	4	356	1.4	254	1425
C	Roof tube 2	3	137	1.4	98	411
D	Roof tube 2	7	92	1.4	66	643
E	Roof tube 2	3	223	1.4	159	668
F	Floor tube 1	5	230	1.2	191	1149
G	Floor tube 1	10	274	1.2	228	2736
H	Floor tube 1	4	281	1.2	234	1122
H	Floor tube 2	3	172	1.2	144	517
I	Floor tube 2	7	113	1.2	94	794
J	Floor tube 2	3	133	1.2	111	399
A	Outer wall 1	2	247	1.2	206	494
K	Outer wall 1	5	92	1.2	76	458
F	Outer wall 1	2	199	1.2	166	399
C	Intermediate wall 1	2	196	0.8	246	393
L	Intermediate wall 1	5	99	0.8	124	495
H	Intermediate wall 1	2	264	0.8	330	527
C	Intermediate wall 2	2	182	0.8	228	364
L	Intermediate wall 2	5	100	0.8	125	500
H	Intermediate wall 2	2	209	0.8	262	419
E	Outer wall 2	2	126	1.2	105	253
M	Outer wall 2	5	92	1.2	76	458
J	Outer wall 2	2	134	1.2	112	268

Table H-4 Required amount of reinforcement with scenario 2

Concrete	110	[m³/m]
Reinforcement	18613	[kg/m]
Overall	169	[kg/m³]

Table H-5 Quantaties for scenario 2

Distribution reinforcement is excluded from the determined amount of reinforcement.

Appendix I Application of GFRP rebar

There are several materials that can be used as substitute for reinforcement steel. The most promising material is glass fibre reinforced polymer rebar. There are a few important reasons why this particular material could be beneficial, these are listed below.

- The material has a high tensile strength compared to reinforcement steel.
- The material has a relatively small unit weight compared to reinforcement steel.
- The material can be applied as a substitute for, or in addition to reinforcement steel.
- The thermal expansion coefficient of glass fibre complies very well with that of concrete which makes the material suitable as reinforcement.

Design

There are a few important aspects for the design of concrete structures in which GFRP is used as reinforcement, these are listed below.

- The modulus of elasticity of GFRP is significantly lower than for reinforcement steel. Therefore the occurring deflections will be of larger magnitude.
- Combination with reinforcement steel is possible.
- It is recommended to apply a tensile strength of 435 N/mm² at the ultimate limit state for the European climate. For accidental loads like the event of an explosion, a higher tensile strength may be taken into account.
- Since GFRP is not susceptible to corrosion, the concrete cover can be significantly smaller than for regular steel rebar. The recommended value is the bar diameter + 10 mm, independent of the exposure class.
- The bond of GFRP is similar to reinforcement steel.
- Long term deflection and bond creep factors of GFRP are similar to those of reinforcement steel.
- The resins of the GFRP can withstand temperatures up to 200 ^{o}c. In order to provide sufficient resistance against fire, the concrete cover can be increased or a fire resistant coating can be applied to the concrete surface.
- In order to withstand shear forces, stirrups are available. Furthermore, double headed bolds are developed for this purpose.
- Due to the fact that application of GFRP is not very common, only a limited collection of diameters are available. The variety in diameters and accessories increases continuously however. Besides, it is possible to acquire those custom made.
- In case exclusively GFRP will be applied, crack widths may be relatively large since corrosion is not an issue.

Application for explosion resistant immersed tunnel

For the design of an explosion resistant tunnel, partly substitution of the excessive amount of reinforcement steel by GFRP may be beneficial. It should be noted that the tensile strength of the material is relatively high. For extreme loads, the compressive strength of the concrete may become critical however. The cross-section should in that case be increased to provide sufficient capacity, resulting in difficulties with respect to the buoyancy of the element. For loads that can be accommodated by means of regular reinforcement steel, there are advantages. Glass fibre reinforcement has a high tensile strength, so fewer bars are required. Besides this the self weight of the bars is small which is advantageous for the execution phase. In order to limit the deflections of the structure during daily circumstances, regular rebar can be used to withstand the permanent loads originating from the water and soil pressure. The additional required reinforcement can be partly accommodated with GFRP. The intermediate walls are for instance not loaded in horizontal direction under normal circumstances. Therefore the application of GFRP may be attractive for these elements. The locations where the GFRP may be beneficial are indicated with green in the figure below.

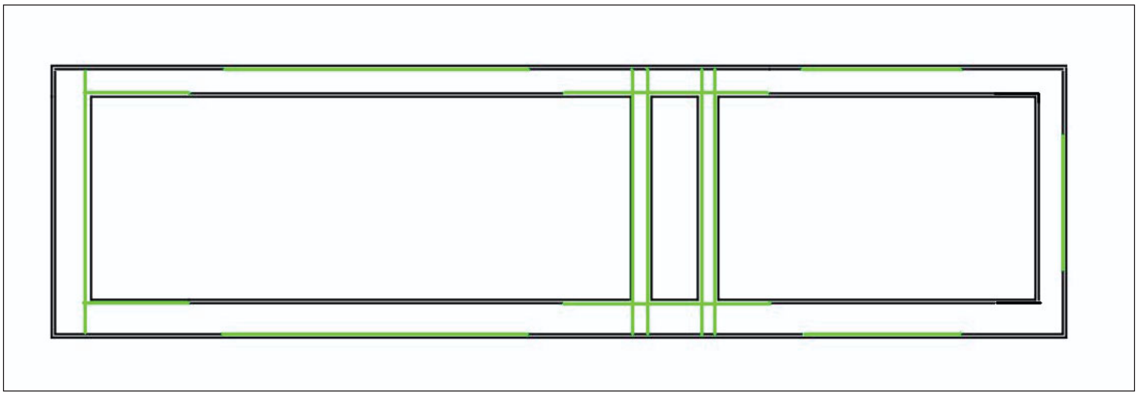


Figure I-1 Possible location GFRP

In figure I-1the locations where reinforcement should be applied for a regular tunnel project are indicated. Since an explosion can be schematized by means of an outward directed load and an inward directed load, reinforcement is required at the top and bottom of each element. In figure I-1 it is indicated where GFRP can be applied anyway. Especially the locations where only reinforcement is required in order to withstand the high explosion load are of interest in this respect. Since the material behaviour of GFRP is linear elastic till failure, there will be no permanent deformations as a result of an explosion at these locations. Fire resistance may be a problem for the GFRP, for the locations at the outside of the cross-section there is a thick layer of concrete that protects the reinforcement.

Execution phase

Special attention should be paid to the execution phase for the application of GFRP as a substitute for, or addition to reinforcement steel. These aspects will be discussed in the following.

- Long lasting exposure of GFRP to ultra violet rays should be avoided, since this may result in deterioration of the material. It is however determined that no reduction in the performance of the material occurs if it is unprotected and exposed to the European climate for 8 weeks.
- Lifting the GFRP by means of a crane, results in deflections that are of the same order of magnitude as for steel reinforcement. The application of cross beams is therefore necessary for long bars.
- Due to the small deadweight of the GFRP, transport and handling are easy compared to steel reinforcement. Besides this, fewer bars are required as a result of the significantly higher tensile strength.
- Cutting of the GFRP is relatively easy compared to steel. Use of a hacksaw or grinder with diamond blade is recommended by the manufacturer for this purpose.
- The strength of the glass fibres perpendicular to the bars is only limited, therefore impact forces should be avoided and the bars should be handled with care.
- Dragging the GFRP on the ground may result in damage to the ribs, which may result in a decreasing bond. Therefore this should be avoided.
- Assembling of the bars should be preferably done by means of regular tying wire.

- It is possible to connect GFRP to regular steel reinforcement. This can be done by means of wire rope grips, whereby a clamped connection is realized. Alternatively a bar coupler can be glued to the GFRP, which makes it possible to connect a steel bar that is provided with thread.

- Bending of the GFRP can not be done on site. The bars have to be bended in the factory in a special process. Nearly all bending forms that can be performed for steel reinforcement can be realized.

- Attention should be paid, if welding takes place in the vicinity of GFRP. Especially if the GFRP is coupled to regular steel rebar that needs to be welded.

- Due to the small deadweight of the GFRP, the material may tend to float up during the compacting of the concreted by means of a vibrating needle. This should be prevented by providing a suitable connection of the bars to the spacers and adjusting blocks by means of tying wire.

- It should be noted that there is only limited experience in the application of GFRP, since only a few projects are realized so far.

It can be concluded that the application of glass fibre reinforced polymer rebar may be an efficient substitute for regular steel rebar for situations in which the required structural capacity also could be provide with steel. The maximal capacity of a cross-section can not be increased, since the compressive capacity of the concrete becomes the limiting factor. The advantages of GFRP are primarily connected to the execution, since the bars has relatively high strength and low density. For the requirement that is stated for the Oosterweel tunnel it could be a beneficial to partly use GFRP.

Appendix J Exploring calculations sandwich structure

The effectiveness and applicability of a sandwich structure is investigated, the results of this study are presented in the following.

The principle of this solution is to provide the tunnel with a secondary tube and fill material which accommodates the required resistance against an explosion. It is attempted to investigate the effect of this solution by means of dynamic calculations, using Plaxis. The starting points and modelling are similar to the analyses performed in paragraph 4.5.3. The hardening soil material model was applied and the surrounding soil is schematized as Pleistocene sand with the following properties.

γ_{unsat}	17	[kN/m³]
γ_{sat}	20	[kN/m³]
E_{50}^{ref}	$4 \cdot 10^4$	[kN/m²]
E_{oed}^{ref}	$4 \cdot 10^4$	[kN/m²]
E_{ur}^{ref}	$1.2 \cdot 10^5$	[kN/m²]
Power	0.7	[-]
C_{ref}	1	[kN/m²]
φ	31	o
ψ	0	o
$\gamma_{0.7}$	$1 \cdot 10^{-4}$	
G_0^{ref}	$1.5 \cdot 10^5$	[kN/m²]
R_{int}	0.9	-
α	0.001	
β	0.0015	

Table J-1 Soil data

Since it is not clear what the effectiveness of this principle will be, some exploring calculations are made. Obviously the effect of the sandwich structure is dependent on the dimensions and material used for the secondary type and fill. For these exploring calculations it is decided to investigate a sandwich structure with a reinforced concrete secondary tube. Furthermore, the fill material is assumed to be sand initially.

The cross-section for the second Coen tunnel as developed by BAM Infraconsult will be used as a starting point. To start with, the following configuration will be considered.

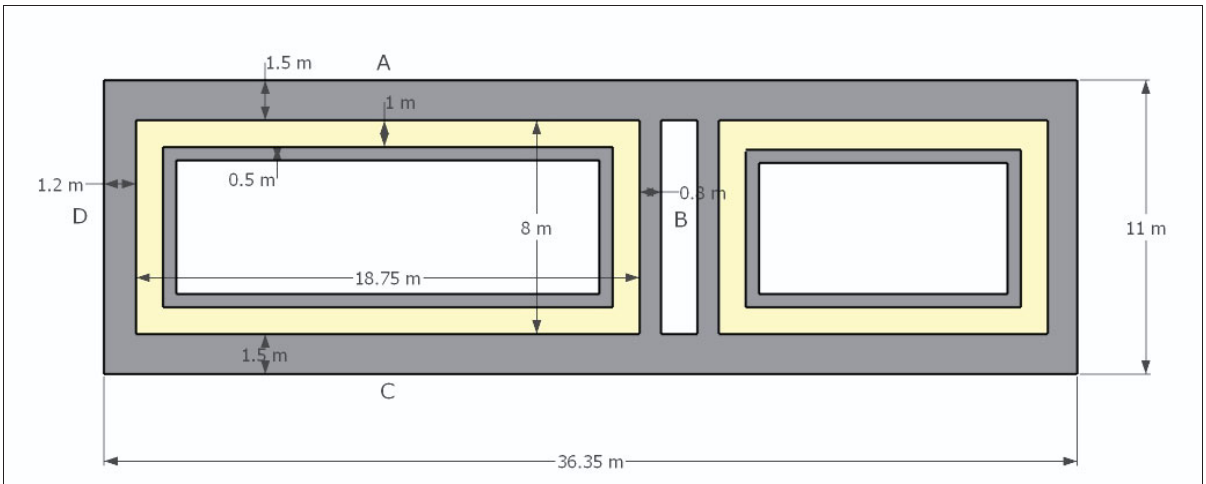


Figure J-1 Considered lay-out

It should be noticed that the dimensions of the element are considerably increased compared to the regular cross-section. As a result of the larger height and spans, the thicknesses of the primary tube are increased somewhat. Besides this, the primary tube should have some extra capacity to resist the explosion load partly.

The stability during transport is checked. The freeboard for just the primary tube is approximately 1.3 m. It is possible to apply part of the sandwich structure initially however.

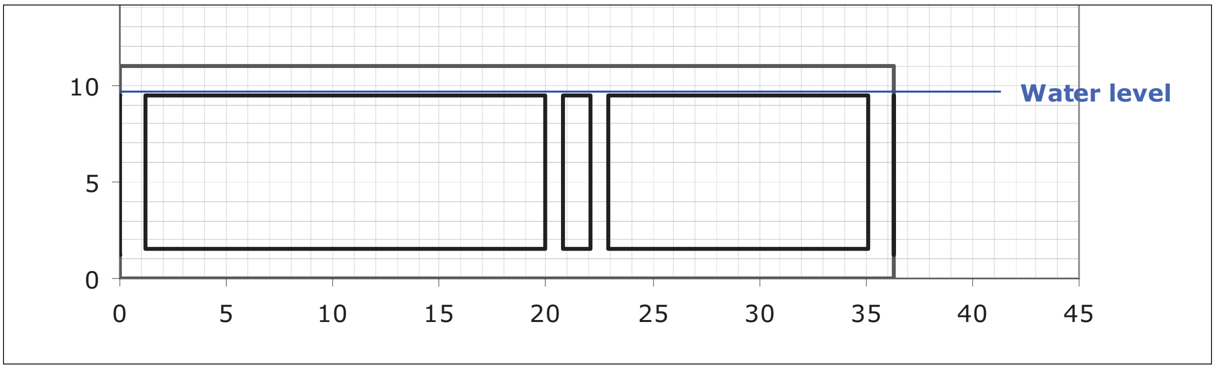


Figure J-2 Stability check primary tube in transport phase

The explosion load will be applied to the large tube, since the effect will be most striking there.
The deformations of the primary tube, for the locations indicated in figure J-3, are calculated with Plaxis. The results are presented in the figure below.

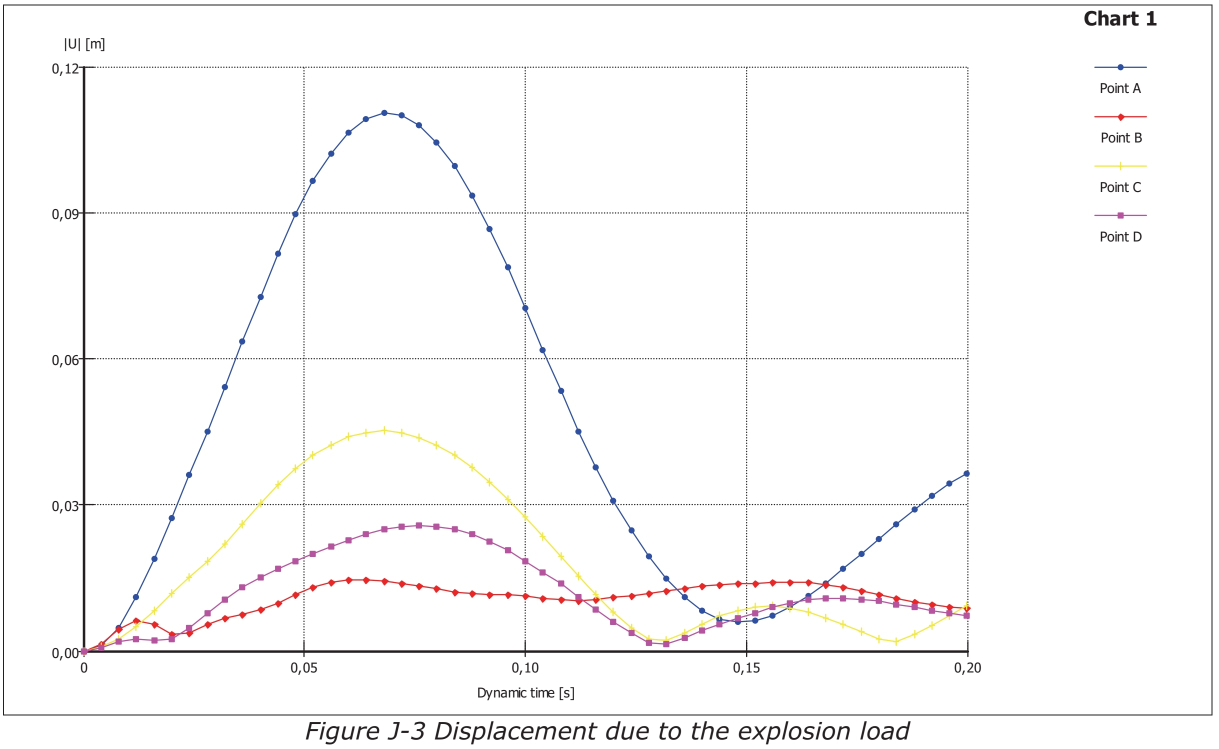


Figure J-3 Displacement due to the explosion load

It can be concluded that large deformations occur. Furthermore it is observed that the plastic bending moments are exceeded at several locations. For the secondary tube failure occurs, which is acceptable.

Adaptations are required to provide sufficient capacity.In order to limit the height of the element, it is decided to increase the thickness of the floor and apply the sandwich structure just for the walls and roof slab. Furthermore, the thickness of the secondary tube for the roof slab will be increased to 1 m.

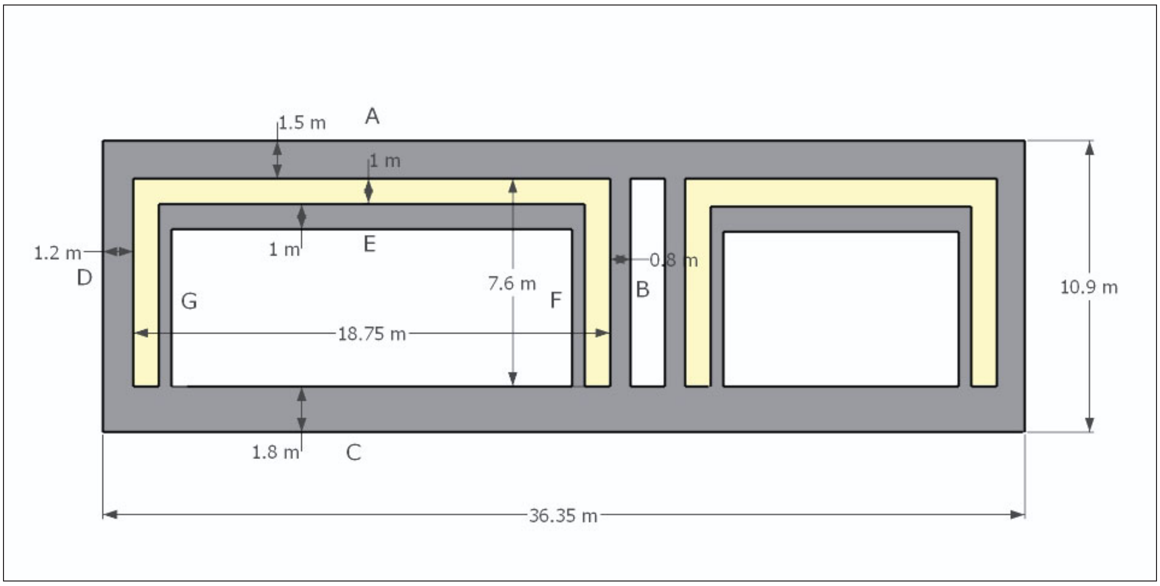


Figure J-4 Adapted cross-section sandwich structure

From the stability calculations it is concluded that the freeboard of the primary tube is approximately 0.63 m.

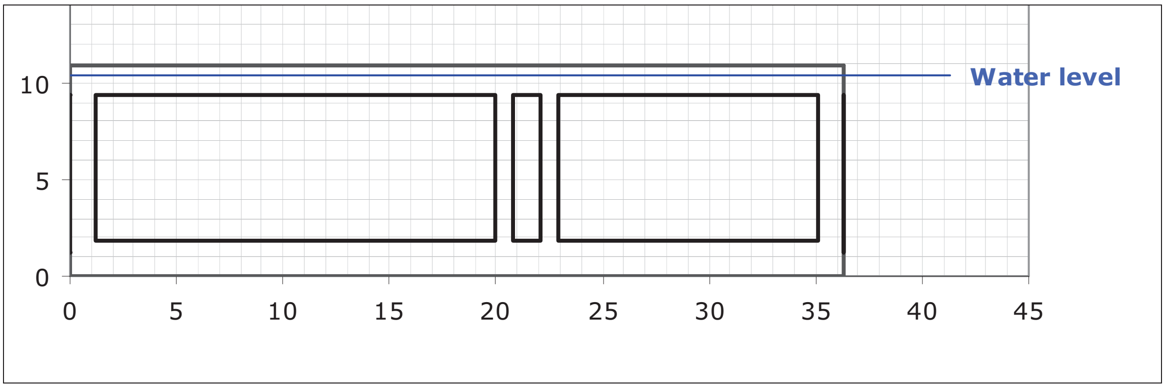


Figure J-5 Stability check primary tube adapted cross-section

In the figures below, the displacement perpendicular to the elements of the secondary tube and primary tube are displayed. The considered points are indicated in figure J-4 The displacement of the primary tube is printed solid blue, whereas the accompanying displacement of the secondary tube is printed dotted in red.

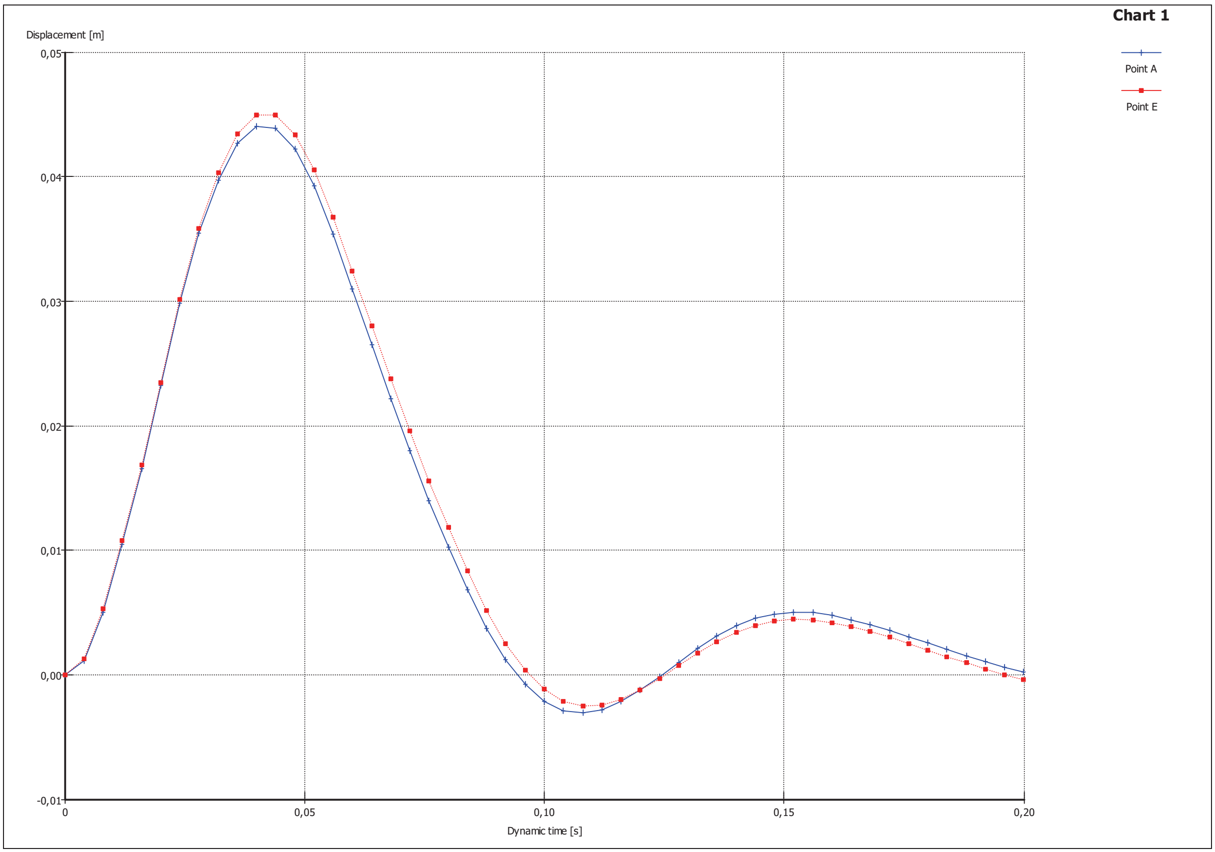


Figure J-6 Displacement of the primary and secondary roof slab with sand fill

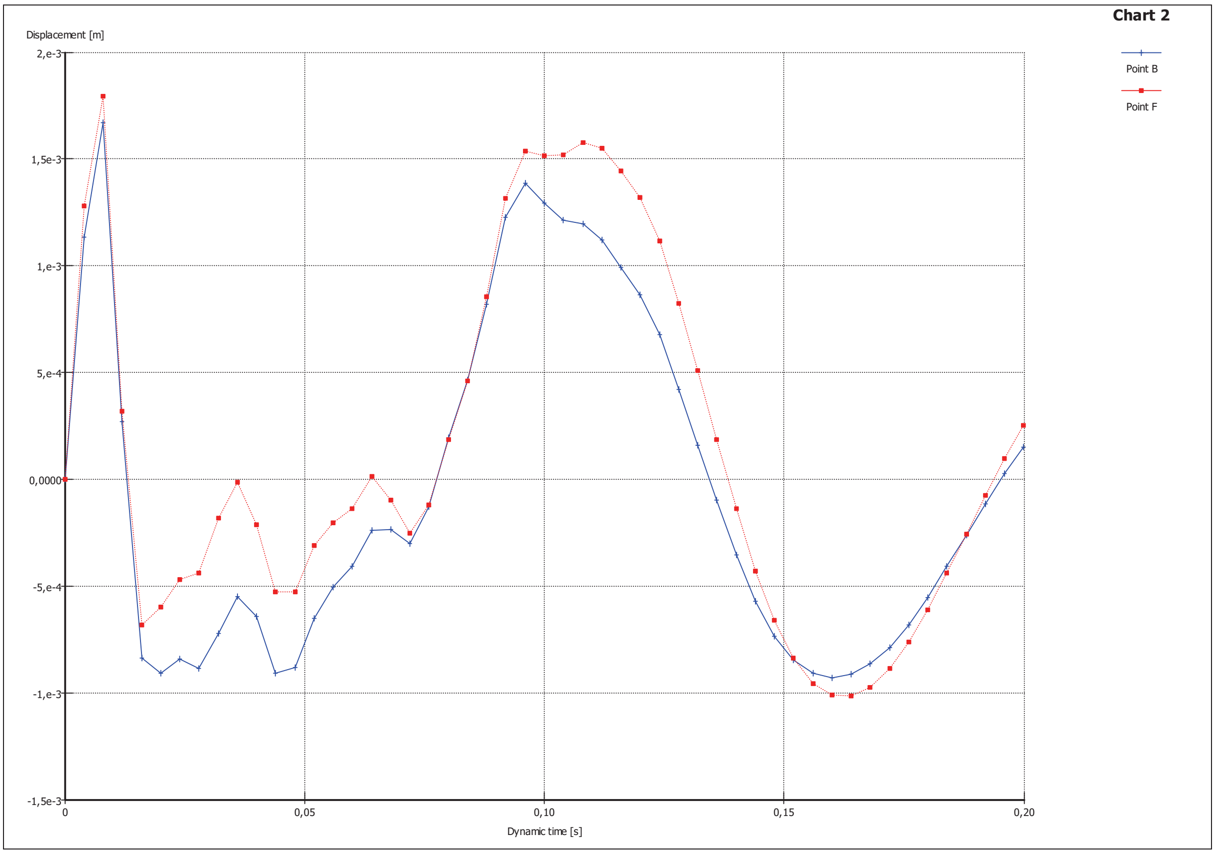


Figure J-7 Displacement of the primary and secondary intermediate wall with sand fill

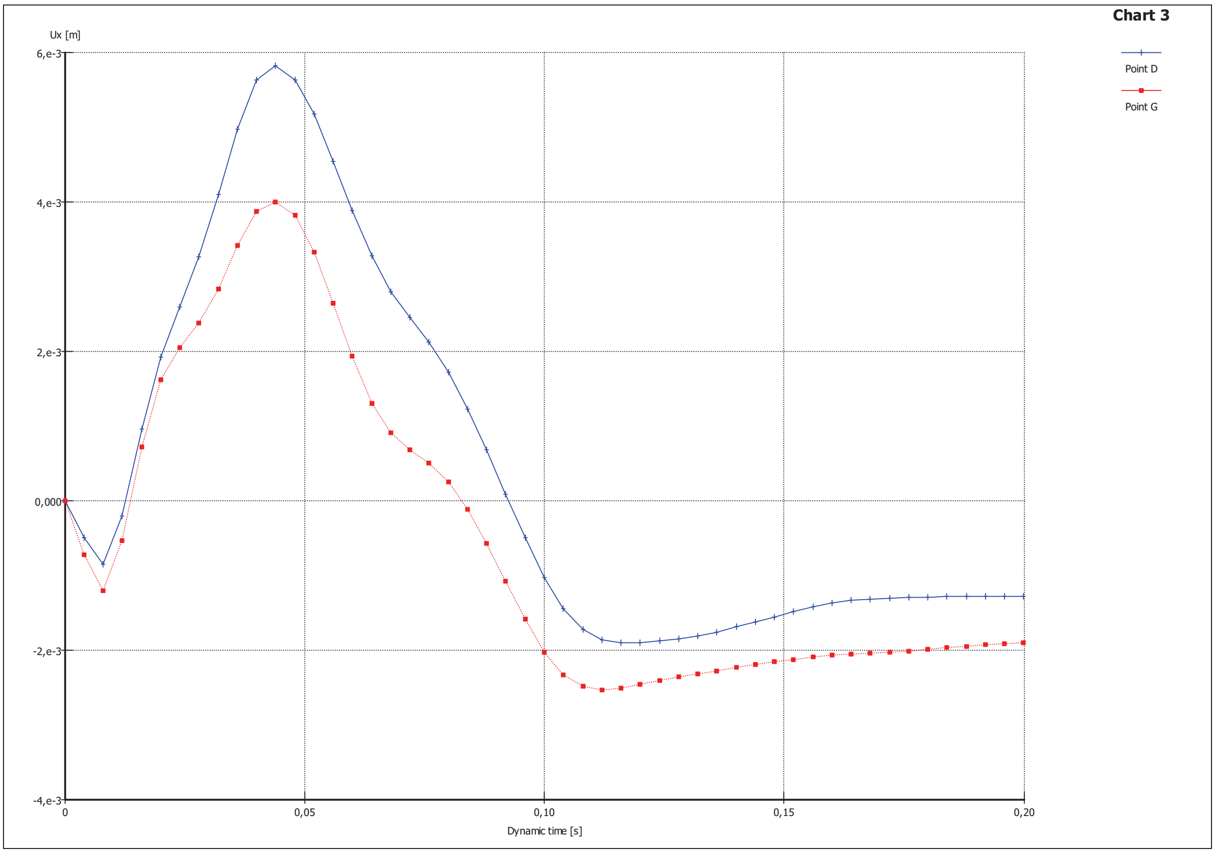


Figure J-7 Displacement of the primary and secondary outer wall with sand fill

It should be noted that the effect of the sand fill is very limited. For the outer wall the deformation of the primary tube is even larger than for the secondary tube. This is probably the result of interaction between the other elements of the primary tube.

The deformations are considerably smaller due to the adaptations. The order of magnitude of the displacements is acceptable. The envelope diagrams for the occurring bending moments, shearforces and axial forces are presented in the figures below.

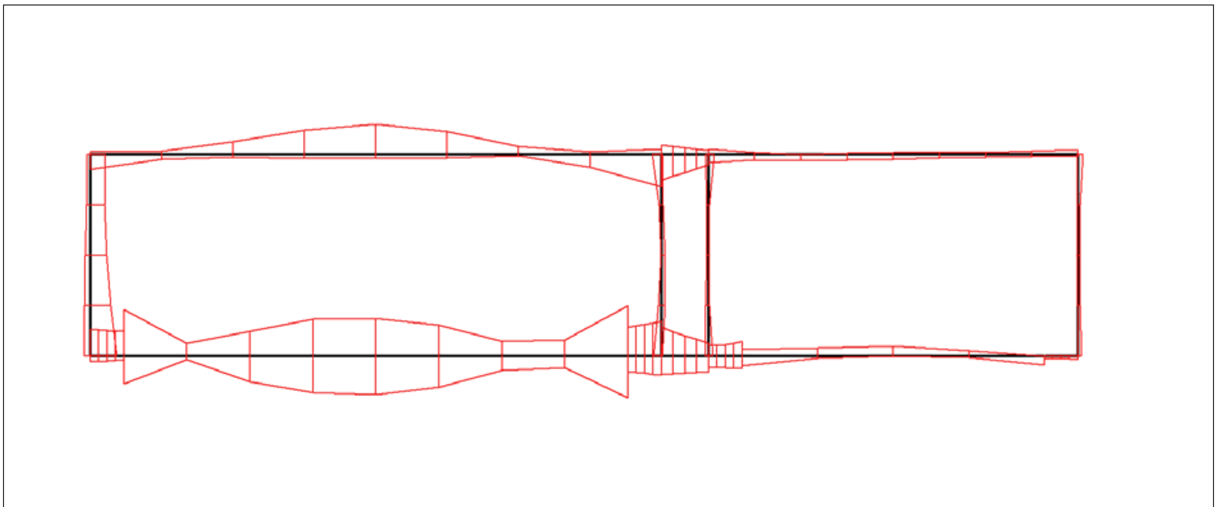


Figure J-8 Envelope diagram bending moments for the primary tube

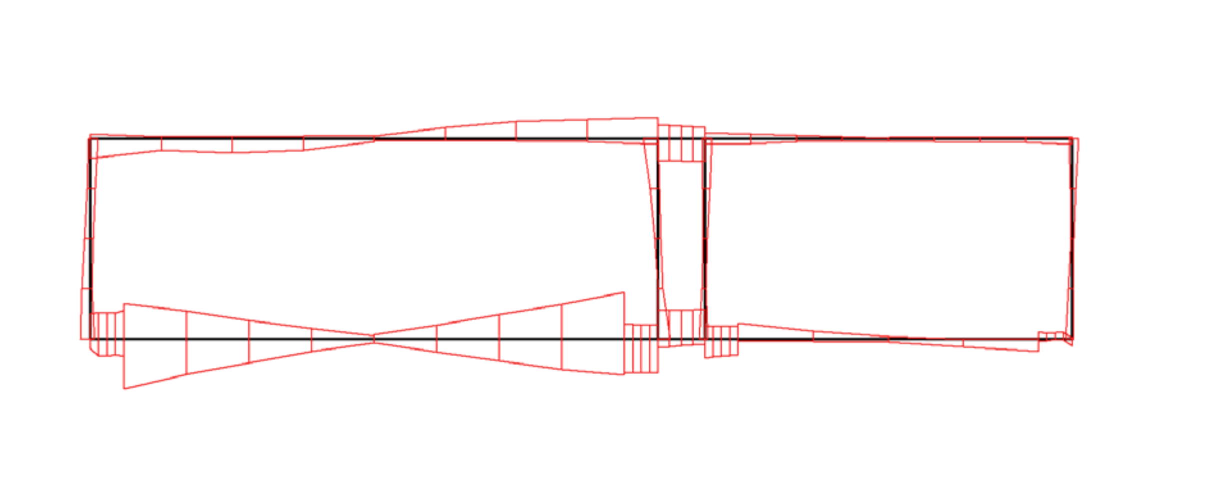


Figure J-9 Envelope diagram shear forces for the primary tube

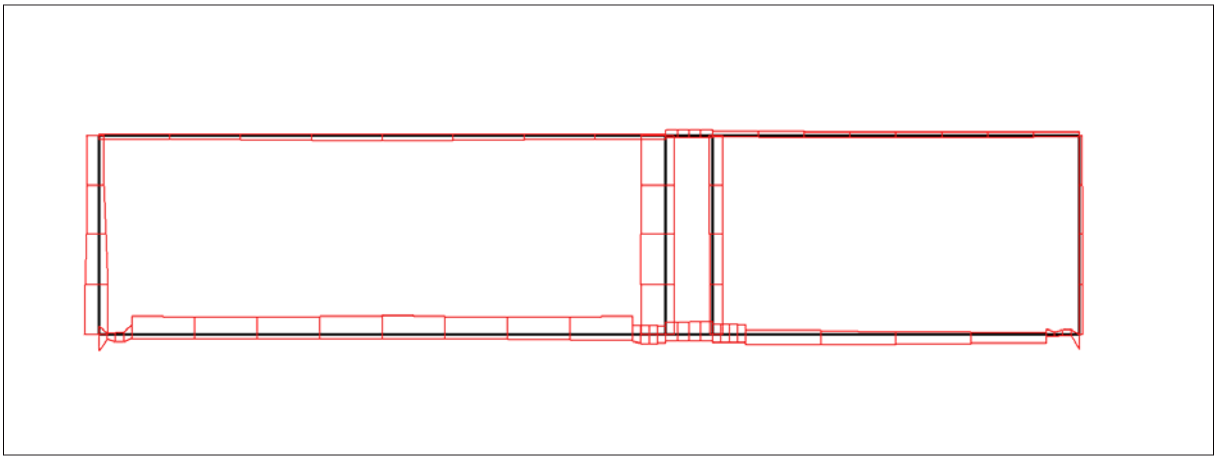


Figure J-10 Envelope diagram axial force for the primary tube

The plastically bending moments for the primary tube are not exceeded for this configuration.

This configuration provides sufficient structural capacity. For the smaller tube, the sand fill as well as the secondary tube could be of smaller magnitude obviously. It could also be considered to not apply a sandwich structure for the smaller tube. Since the span is much smaller, the capacity can be achieved easier there.

By trail and error it is observed that increasing the thickness of the sand layer hardly results in a reduction of the maximal displacements of the primary tube. Sand may however not be the most suitable material to provide dissipation. In case the fill material crushes or compresses due to the explosion load, energy will be dissipated. The application of expanded clay pellets for instance may be beneficial in this respect. It is not possible to model crushing material behaviour with Plaxis, since it is assumed that the grains can not be disintegrate in the code of this program. The effect of a relatively compressible material can be investigated however.

In order to investigate the effect of the fill material, a fictive, relatively high compressible material is modelled instead of sand. For this purpose, the soft soil material model of Plaxis is used. In this model a logarithmic relation between the volumetric strain and the mean stress is presumed. As indicated in the figure below.

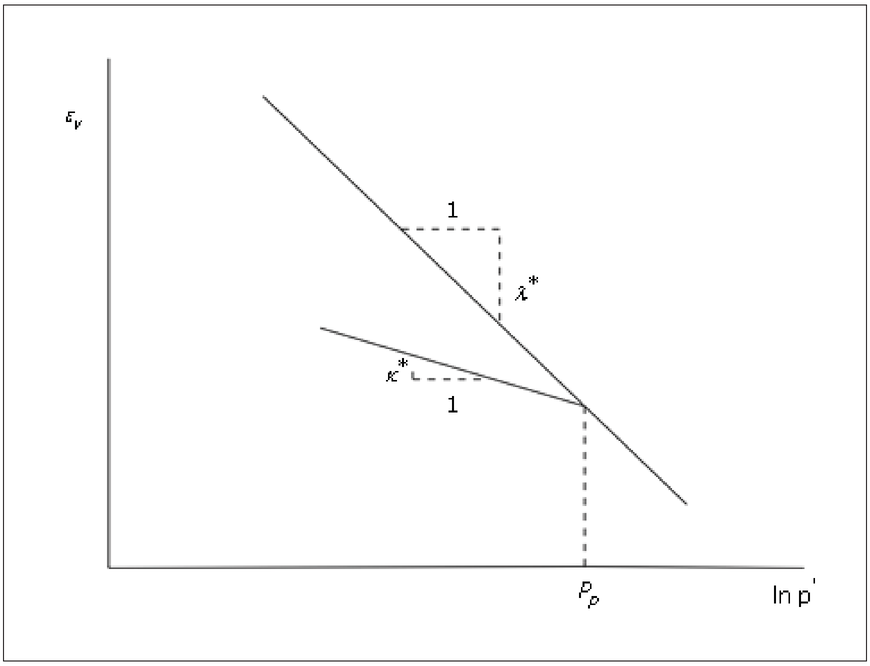


Figure J-11 Strain-stress relation in the soft soil model of Plaxis

In the figure above ε_V is the volumetric strain and $p^{'}$ is the mean stress.
The compressibility and swelling of a material can be described with the following relations.

$$\lambda^* = \frac{C_c}{2.3 \cdot (1+e)}$$

$$\kappa^* = \frac{2C_r}{2.3 \cdot (1+e)}$$

The following parameters were used for this calculation.

Modified compression index	λ^*	2.9
Modified swelling index	κ^*	0.04
Compression coefficient	C_c	10
Recompression coeficient	C_s	0.069
Void ratio	e	0.5

Table J-1 Envelope diagram shear forces

A typical value for C_c of clay is in the order of 1, the fictive material is thus assumed to be 10 times as compressible as clay. This is considered to be an upper limited, besides the characteristics used are not connected to a real material. The purpose of this evaluation is just to investigate the influence of the compressibility of the fill material. The results of the calculation with the fictively compressible fill material are displayed in the figures below.

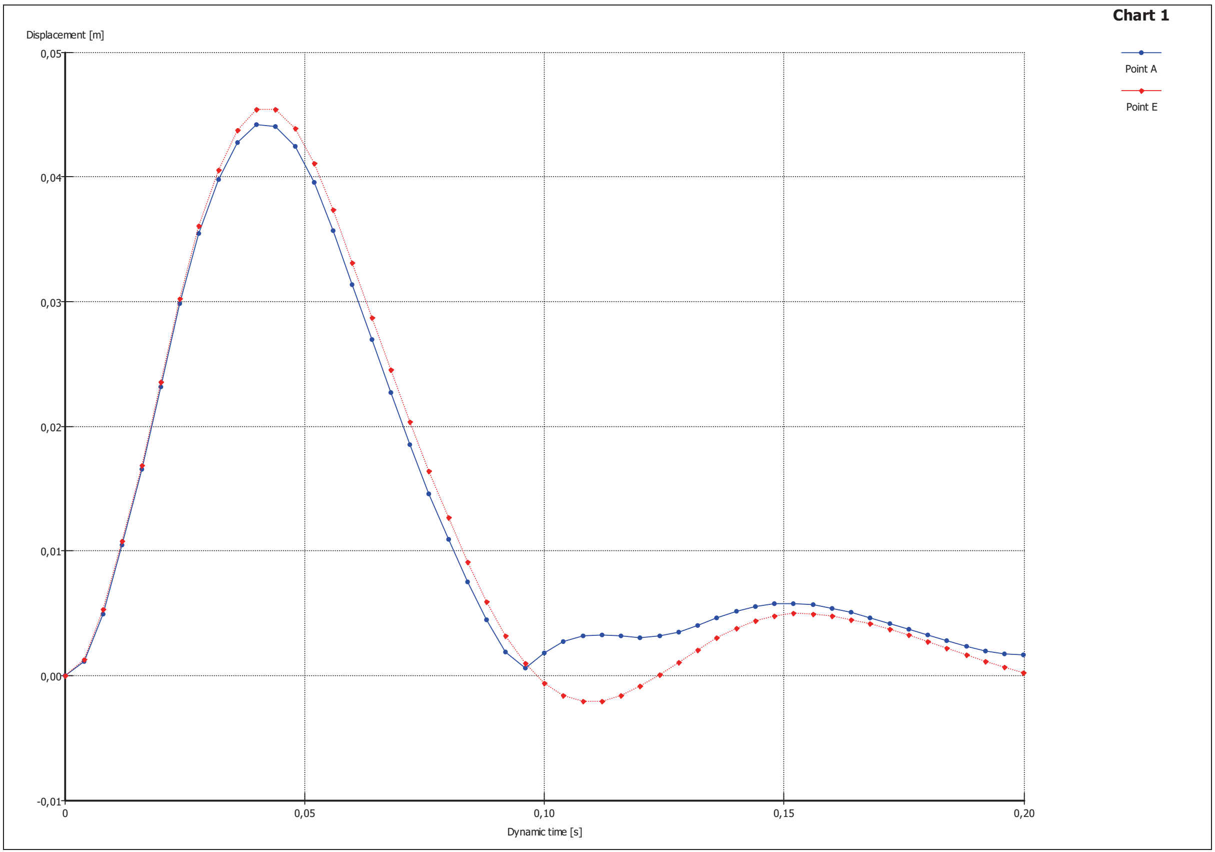


Figure J-12 Displacements primary and secondary roof slab with high compressible fill

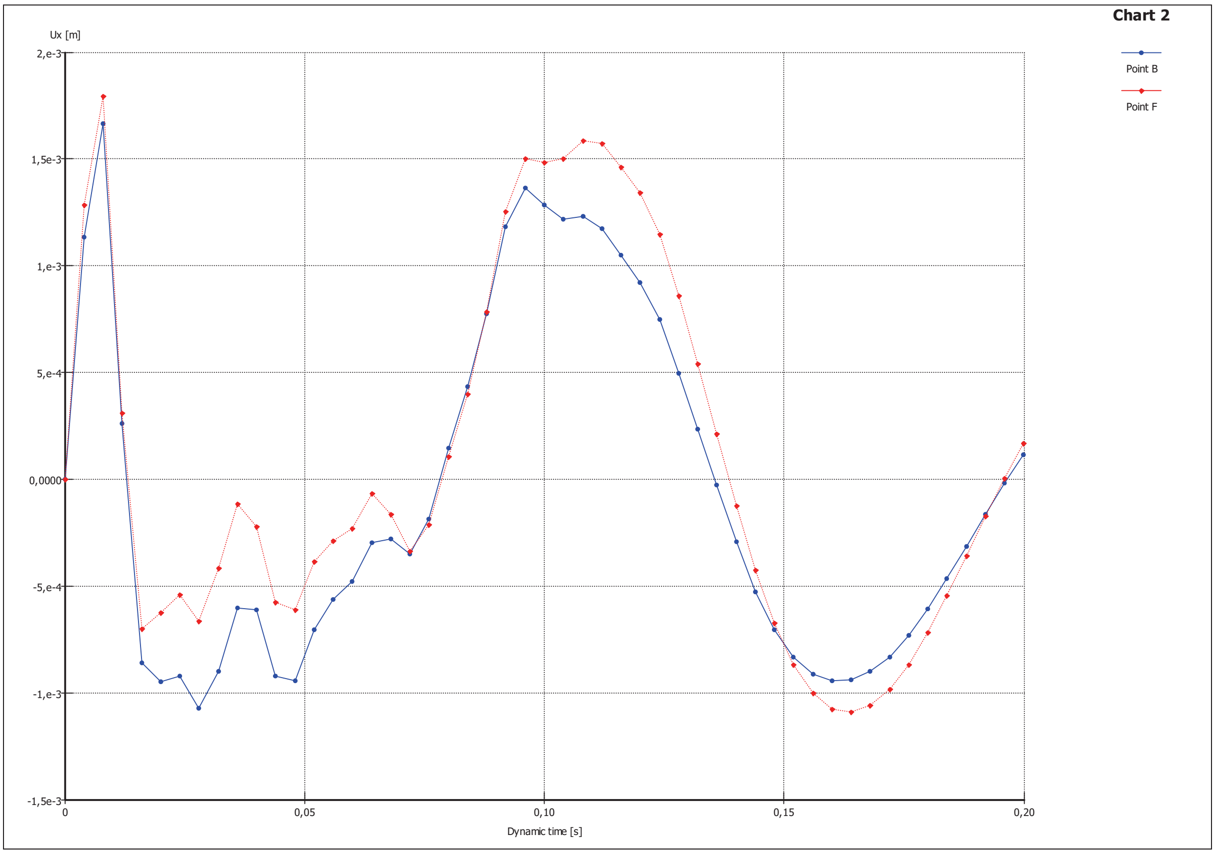


Figure J-13 Displacements primary and secondary intermediate wall with high compressible fill

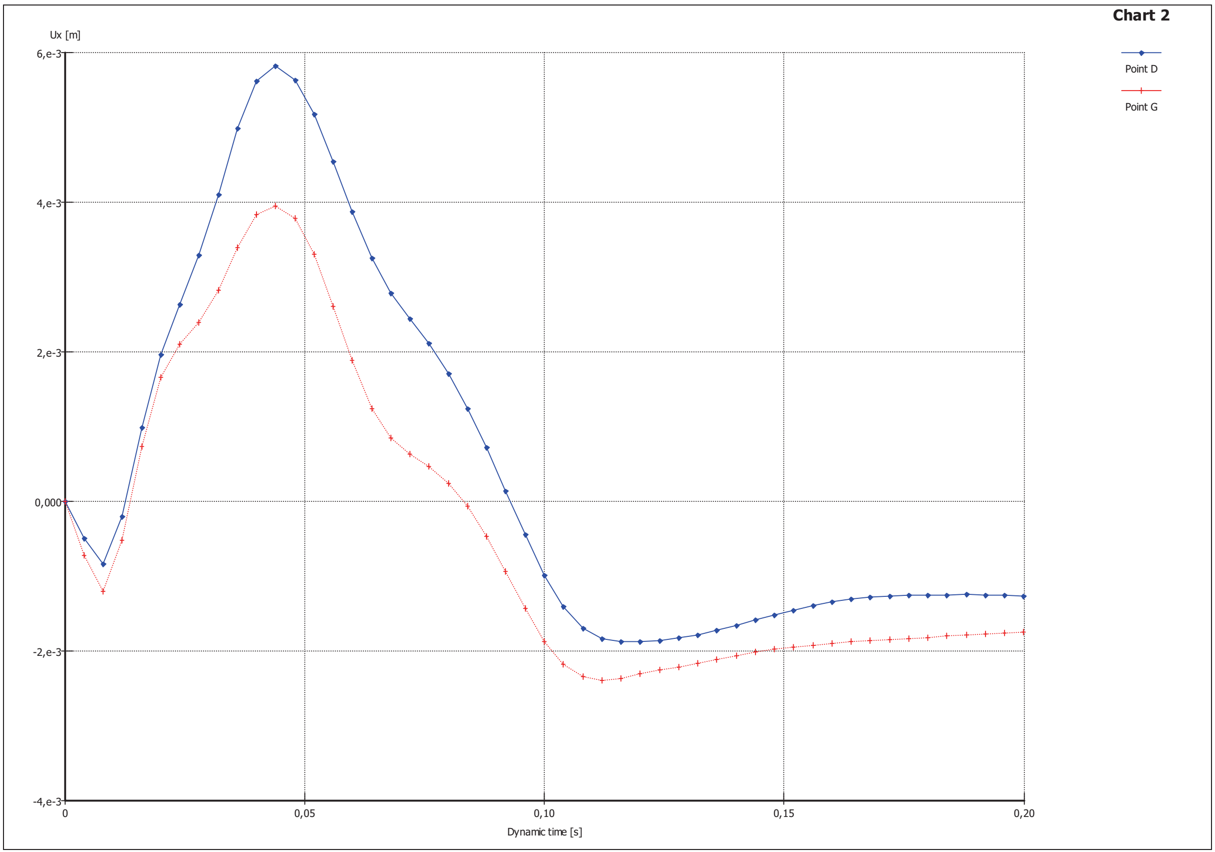


Figure J-14 Displacements primary and secondary intermediate wall with high compressible fill

It should be noticed that there is hardly any difference between the results obtained with sand as fill material and the results for the relatively compressible material.

Conclusion

In the table below, the main dimensions and quantities for the regular cross-section and the sandwich structure as indicated in J-4 are listed. It should be noticed that the sandwich structure is not optimized and therefore the numbers just give an indication.

		Regular cross-section	Sandwich structure
Width overall	[m]	29.9	36.35
Height overall	[m]	7.9	10.9
Structural concrete	[m²]	81	150+40 =190

Table J-2 Comparison with regular cross-sectionl

- It should be noticed that the quantities for the smaller tube could be less, therefore the amount of concrete is overestimated. It can however be concluded that the required amount of structural concrete would be more than twice the required amount for a regular cross-section.
- The height of the elements is increased with 3 meters, which results in considerably longer entrances.
- Due to the increased overall dimensions, the volume that has to be dredged increases significantly.
- The dimensions of the element will increase excessively due to required space for the sandwich structure.
- The effect of the sand fill is very small, while it requires relatively large space.
- The effect of a relatively compressible material appears to be very limited as well.

- The large spans of the floor and roof slab require large thickness of the secondary tube.

- For the transport phase it is beneficial that the secondary tube and fill material can be partly applied after immersion. In the final situation the fill material and secondary tube provide stability, which makes the application of ballast concrete unnecessary.

It can be concluded the cross-section has to be increased excessively to provide sufficient capacity. The height of the elements has to be increased considerably, which is unfavourable since it results in long entrances. Apart from that, the extra amount of reinforced concrete that is required is also very large. For larger spans, the disadvantages are of course of more importance. Based on the determined amount of reinforced concrete and the increased height of the element, it is very unlikely that this alternative should be feasible in terms of economics.

Appendix K Exploring calculations special tube

In this appendix, the results of the exploring calculations for three variants of the alternative separate tubes will be presented.

K.1 1 Special tube

With help of the mass spring model, the required thicknesses for the special tube are estimated. Subsequently, the transport phase and final situation are checked by means of a spreadsheet. A few adaptations are made to the cross-section in order to comply with al requirements.

Since the special tube is of relatively large dimensions, the stability of the element during transport may be a problem. The horizontal position of centre of gravity of the element is not located in the middle of the element. Therefore the element will tilt till the centre of the buoyancy force is located at the same horizontal position as the centre of gravity of the element.

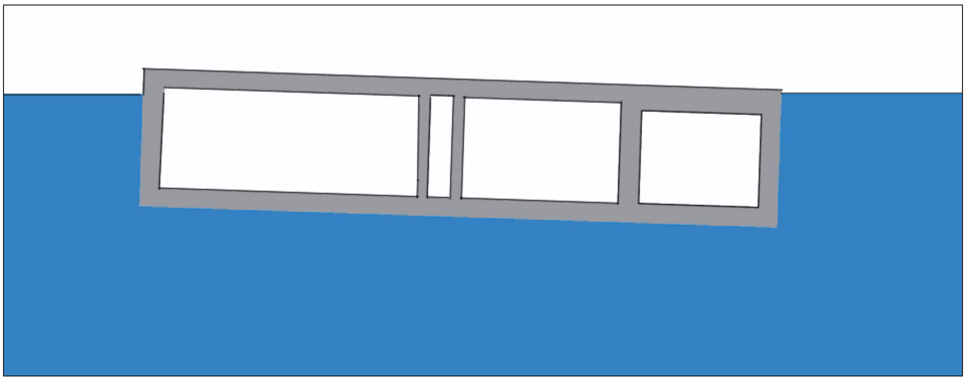


Figure K–1 Tilting of the element

Tilting of the element will not occur if the horizontal position of centre of gravity of the element is exactly halfway the width, since it than is at the same position as the centre of the displaced water mass. In order to achieve this, it could be considered to initially apply some ballast concrete in order to stabilize the floating element. It should be noticed that the draught of the element will increase in that way, and therefore adaptations of the cross-section are required in order to facilitate the minimal freeboard of 0.5 m that is required for transport at sea. By means of trial and error, the following solution is obtained whereby the cross-section as developed for the second Coentunnel is used as a starting point.

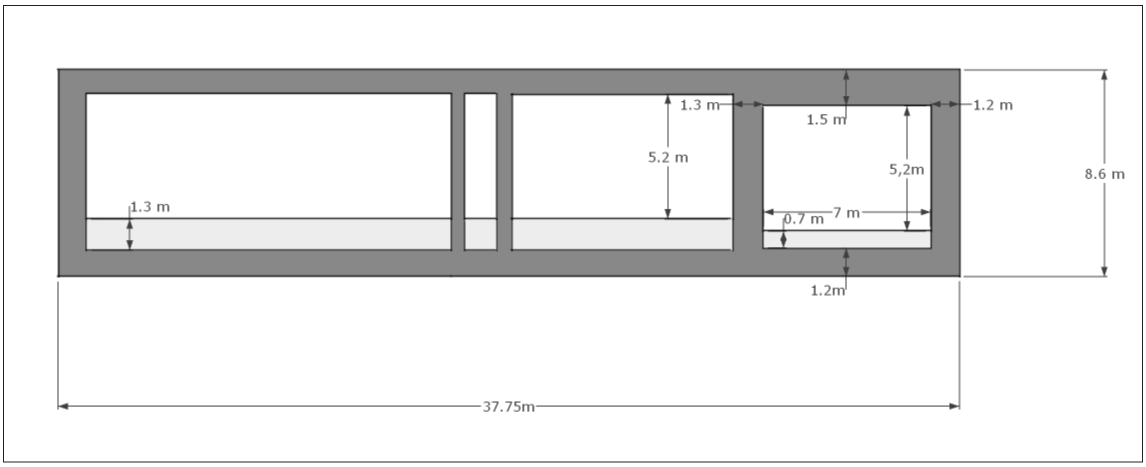


Figure K-2 Dimensions for 1 special tube

Initially the ballast concrete in the large tube can be partially applied in order to provide stability during transport. Alternatively, the application of stabilizing pontoons may provide a solution for this phenomenon. In case at both sides a special tube would be attached the stability is obviously no problem.

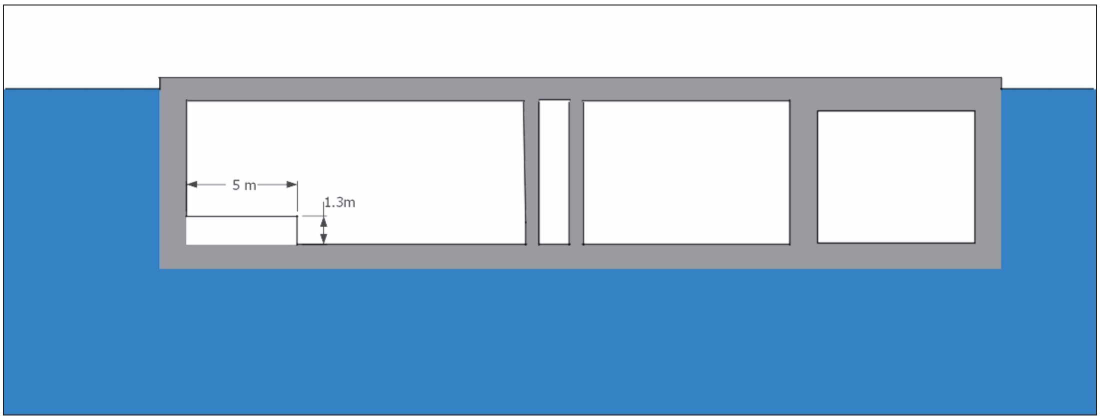


Figure K-3 Application of balast

It is even more efficient to apply water ballast in order to provide stability. A ballast tank should be positioned in the large tube against the outer wall to be most effective, since the lever arm is largest in that case.

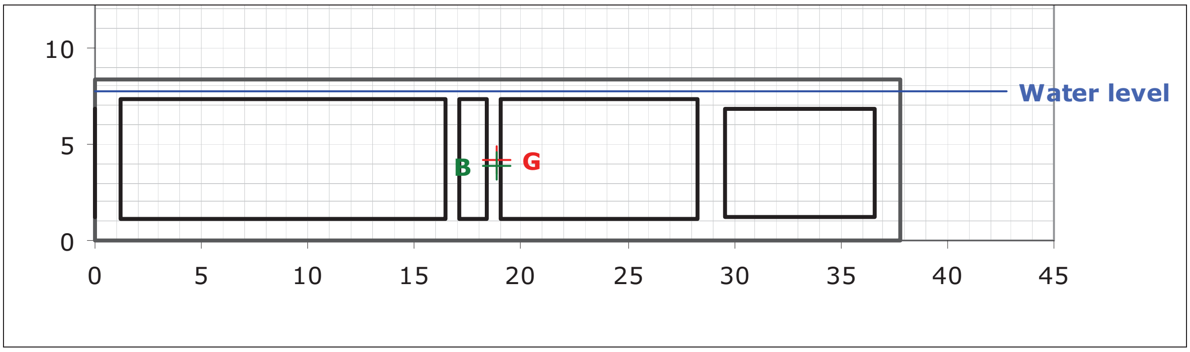


Figure K-4 Stability check of the element during transport

Alternatively, water ballast could be applied, as indicated in the figure below.

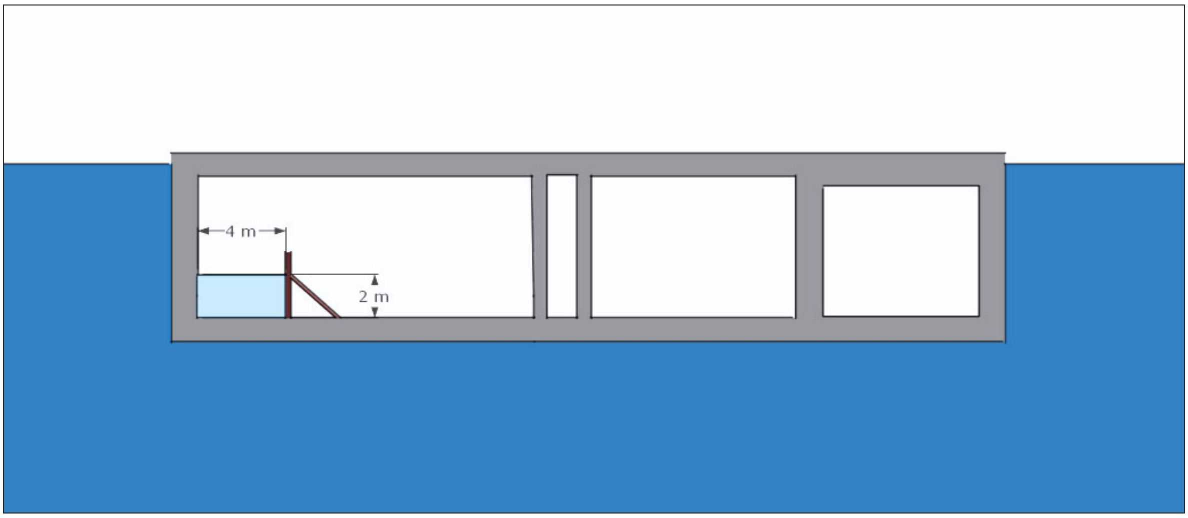


Figure K-5 Application of water ballast

The results of the stability calculations are presented in the table below whereby a section of 1 meter is considered. The centre of gravity of the element during transport is indicated with G the centre of the buoyancy force is indicated with B. The ballast is not shown in this picture, though it is taken into account in the determination of the centre of gravity of the element.

Effective height	5	[m]
Thickness roof slab special tube	1.5	[m]
Thickness floor slab special tube	1.2	[m]
Thickness floor slab	1.1	[m]
Thickness roof slab	1	[m]
Balast height special tube	0.7	[m]
Balast height regular tube	1.2	[m]
Thickness outer wall 1	1.2	[m]
Width tube 1	15.25	[m]
Thickness intermediate wal 1	0.6	[m]
Width corridor	1.35	[m]
Thickness intermediate wal 2	0.6	[m]
Width tube 2	9.25	[m]
Thickness seperation wall 2	1.3	[m]
Width special tube 2	7	[m]
Thickness outer wall 2	1.2	[m]
Overall width	37.75	[m]
Overall height	8.3	[m]
concrete total	113.9	[m²]
Water ballast transport	8	[m²]
Ballast concrete final	35.9	[m²]
Bulk heads + immersion equipement	30	[kN/m]
Weight	2926	[kN]
Maximum buoyancy force	3133	[kN]
weight/ max buoyancy force	0.93	[-]
freeboard	0.55	[m]
Safety in final situation	1.13	[-]

Table K-1 considered configuration

The final safety could be further increased if ears would be attached to the floor slab. These ears are loaded by soil if the immersion trench is filled. The weight of this soil provides additional safety for uplift of the element.

The cross-section is evaluated by means of Plaxis. The special tube is exposed to the representative BLEVE load according to TNO. The response of the structure to this load and the occurring forces are calculated. The results will be discussed in the following.

For the elements of the special tube, the following properties were applied. The plastically bending moment and axial forces are estimated in correspondence to the approach in paragraph 4.5.3.

Property	Roof	Floor	Outer wall	Int. wall	Unit
EA	4.65 E7	3.72 E7	3.72 E7	4.03 E7	[kN/m]
EI	8.719 E6	4.46 E6	4.46 E6	5.67 E6	[kN/m²/m]
d	1.5	1.2	1.2	1.3	[m]
w	25	25	25	25	[kN/m/m]
Mp	1.91 E4	7545	7545	8890	[kNm/m]
Np	9787	7830	7830	8482	[kN/m]

Table K-2 Schematisation tunnel structure in Plaxis

A dynamic analysis is made with Plaxis. The surrounding soil is modelled as Pleistocene sand with the following parameters.

γ_{unsat}	17	[kN/m³]
γ_{sat}	20	[kN/m³]
E_{50}^{ref}	$4 \cdot 10^4$	[kN/m²]
E_{oed}^{ref}	$4 \cdot 10^4$	[kN/m²]
E_{ur}^{ref}	$1.2 \cdot 10^5$	[kN/m²]
Power	0.7	[-]
C_{ref}	1	[kN/m²]
φ	31	°
ψ	0	°
$\gamma_{0.7}$	$1 \cdot 10^{-4}$	
G_0^{ref}	$1.5 \cdot 10^5$	[kN/m²]
R_{int}	0.9	-
α	0.001	
β	0.0015	

Table K-3 Soil parameters Plaxis

In the figure below, the displacements after 0.2 s are displayed. For clearness the displacements are scaled up 100 times.

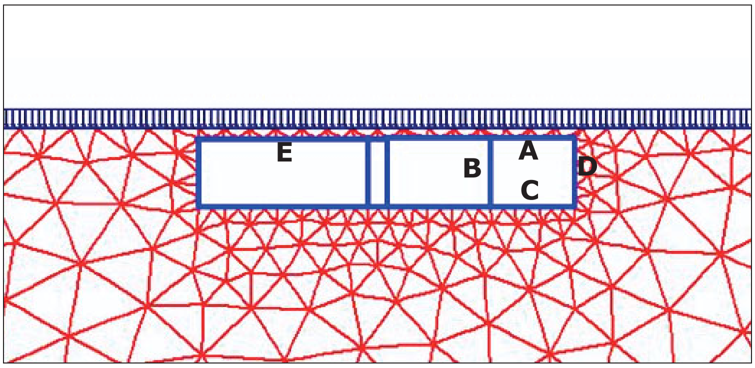
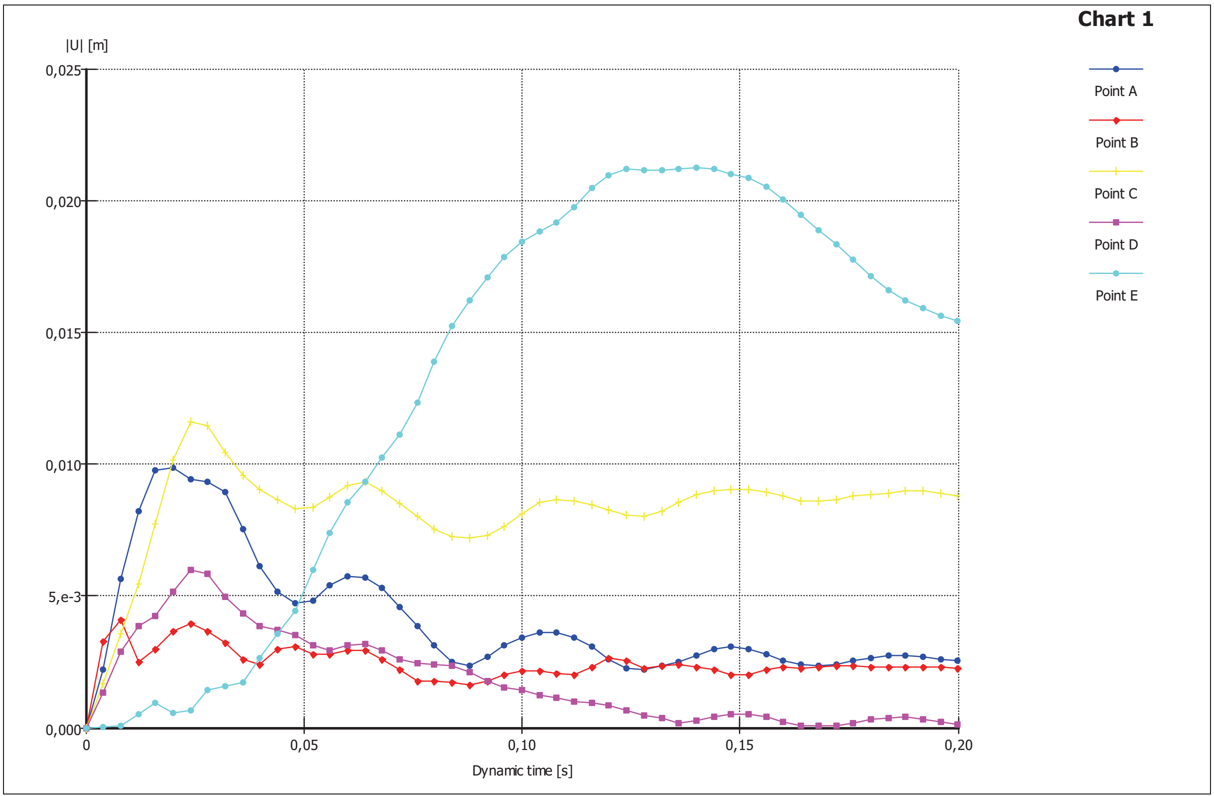


Figure K-6 Displacements aty t= 0.2 s, scaled up 100 times

The displacements of the indicated points as a function of time are presented in Figure. It should be noticed that the largest displacement as a result of an explosion in the special tube occurs in the roof slab of the large regular tube, point E. The order of magnitude of the displacements is however of small magnitude.



FigureK-7 Displacements as a function of time

It should be noticed that the roof slab of the large regular tube (point E) deforms as a result of the explosion in the special tube. All deformations are however within acceptable ranges.

The envelope diagrams for the occurring bending moments, shear and axial forces as a result of the permanent loads and the explosion load between 0 and 0.2 s is presented in the figure below.

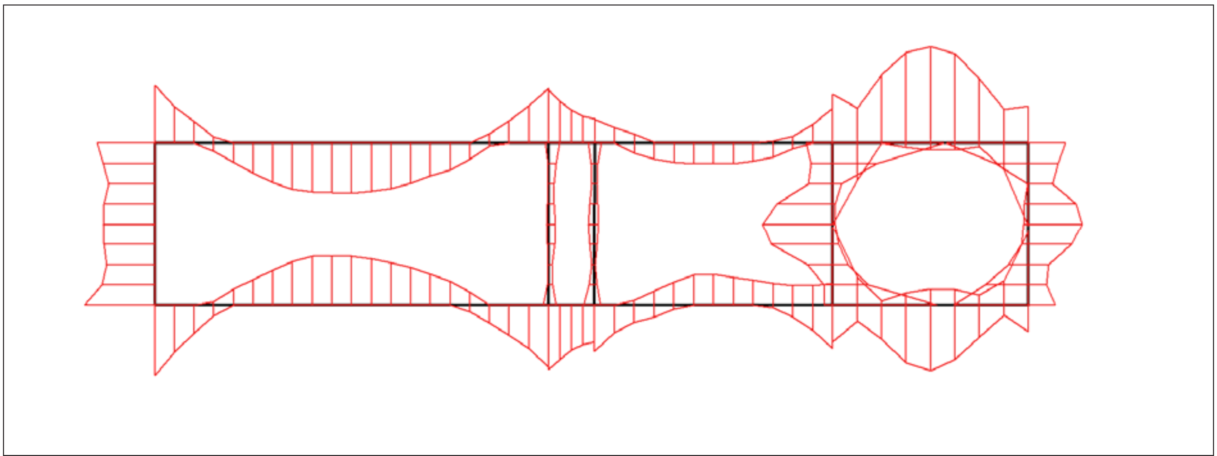


Figure K-8 Envelope diagram bending moments

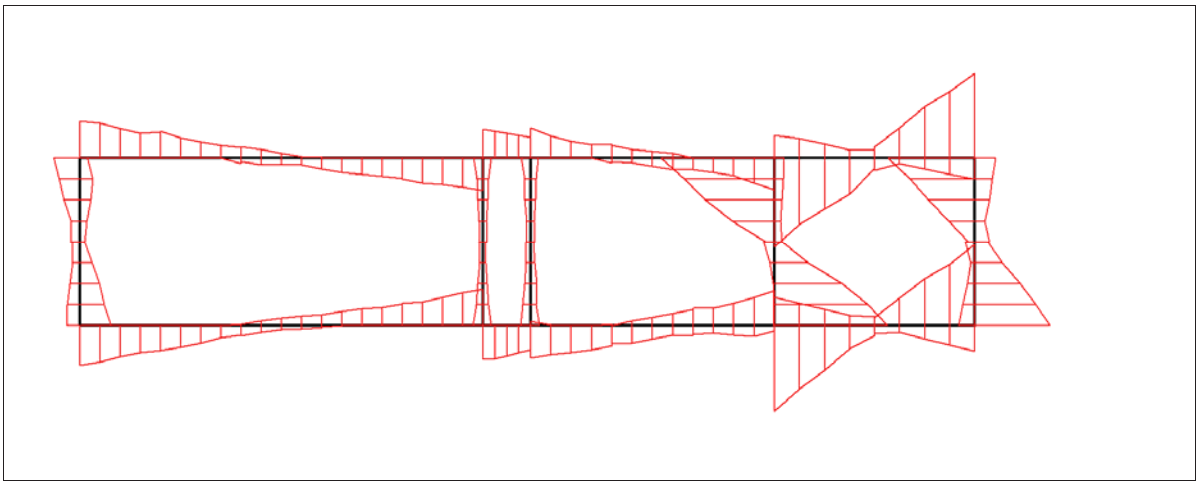


Figure K-9 Envelope diagram shearforces

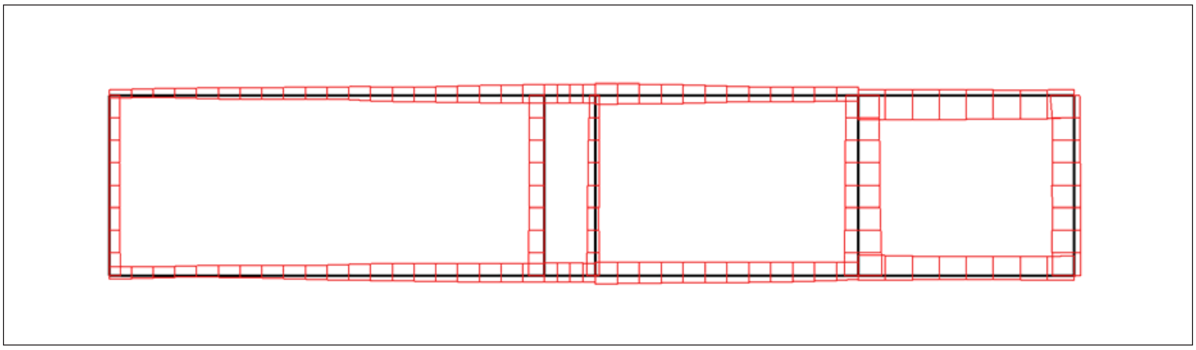
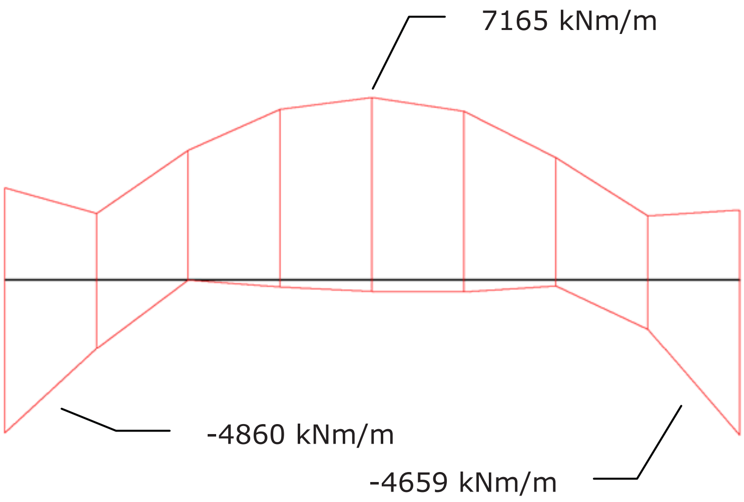


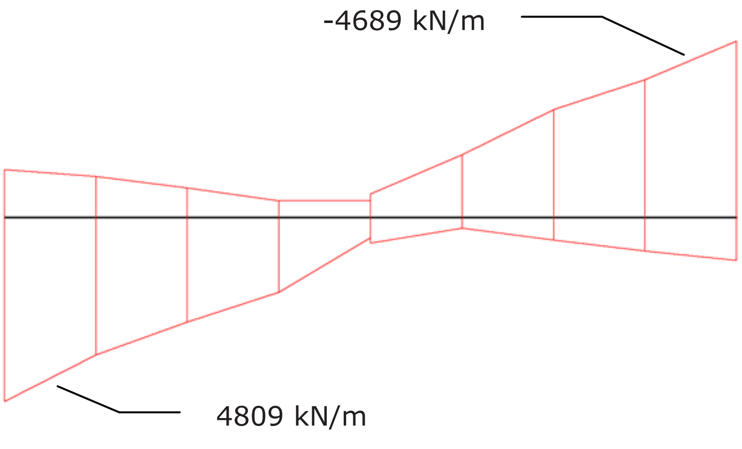
Figure K-10 Envelope diagram axial forces

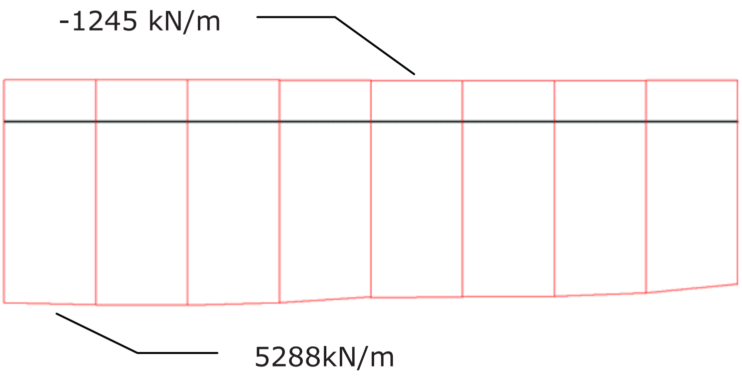
For the regular tube, no deviations of importance compared to the situation without an explosion load are observed. The occurring forces for the special tube will be considered in more detail however.

For each element of the special tube, the envelope diagram of the occurring forces is given, in which the maximum values are indicated. It should be noted that values at the edges of the elements concern the value in the clear of the support, in order to be not too conservative. Besides this, in the final situation there will be a layer ballast concrete that provides additional support for the walls.

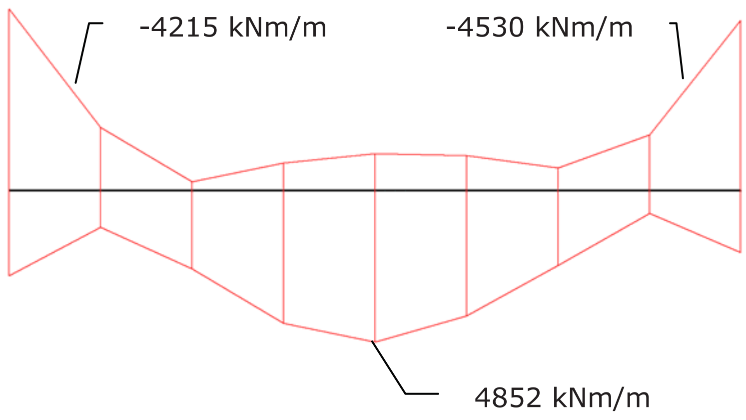
Roof special tube

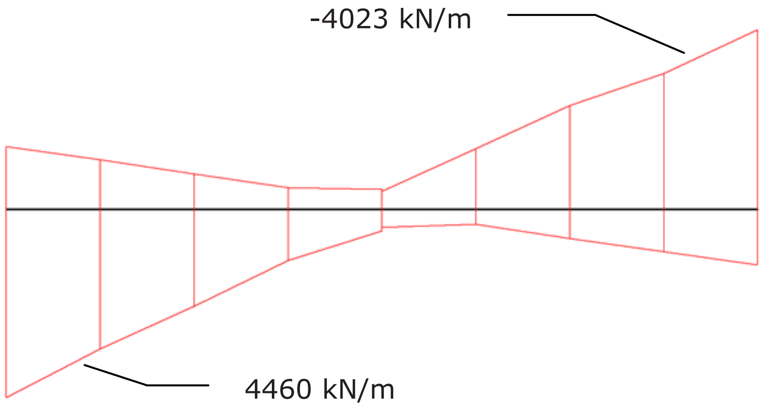


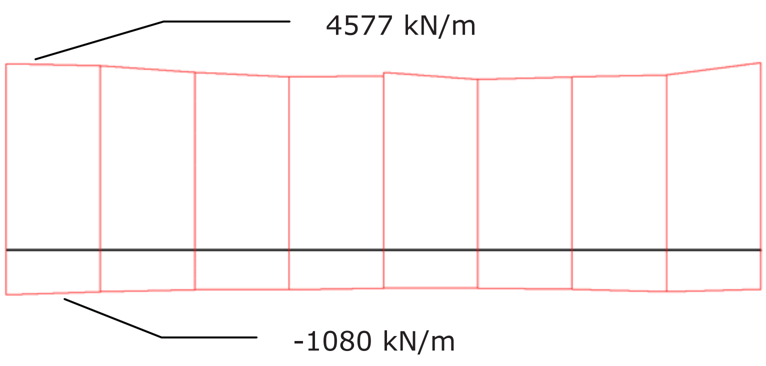




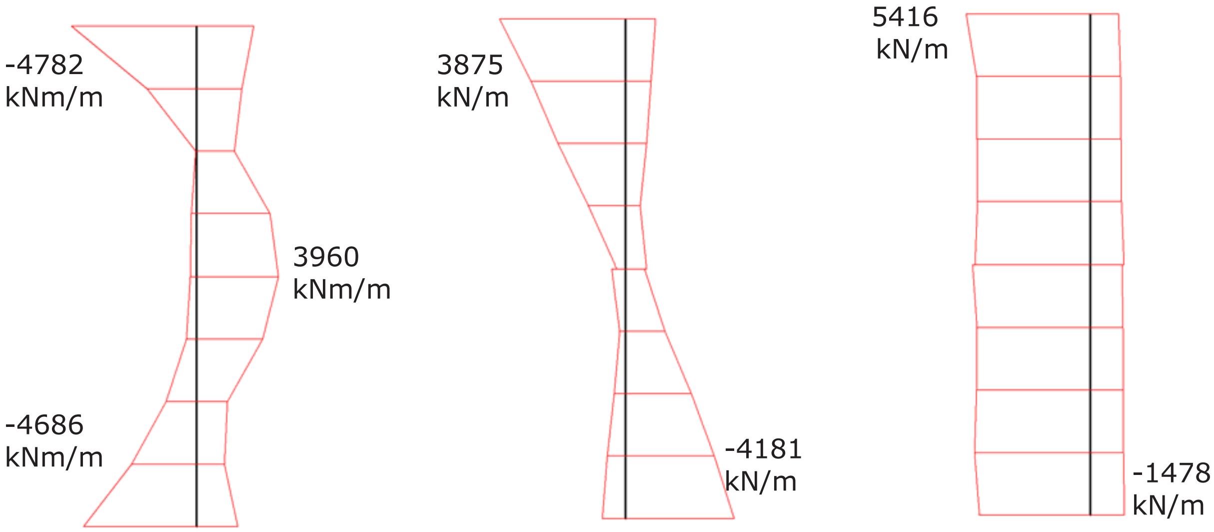
Floor special tube



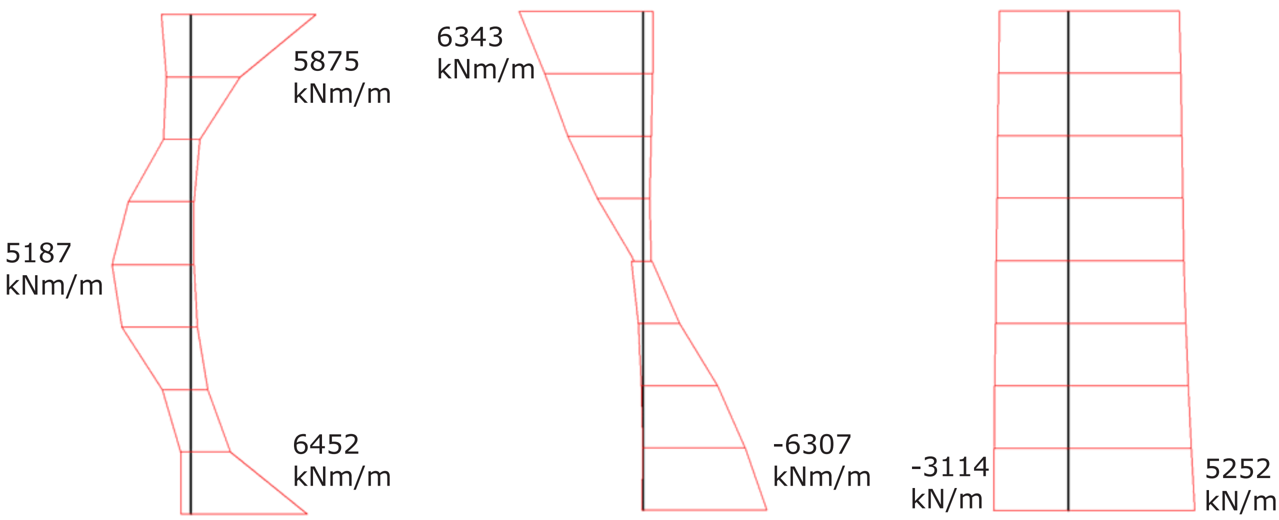




Outer wall



Intermediate wall



From previous figures, it can be concluded that the plastic bending moments were not exceeded. Especially the roof slab could be of smaller dimensions. In all cases there is amply sufficient capacity. Since the plastic bending moments are estimated in a quite simplified manner, this is a good result.

It should be noted that the occurring shear forces are of rather large magnitude. In order to check if it will be a problem to accommodate sufficient capacity to withstand these shear forces, the required amount of bending and shear reinforcement is determined according to the Eurocode, a similar spreadsheet as presented in Appendix F is used to perform the calculations.

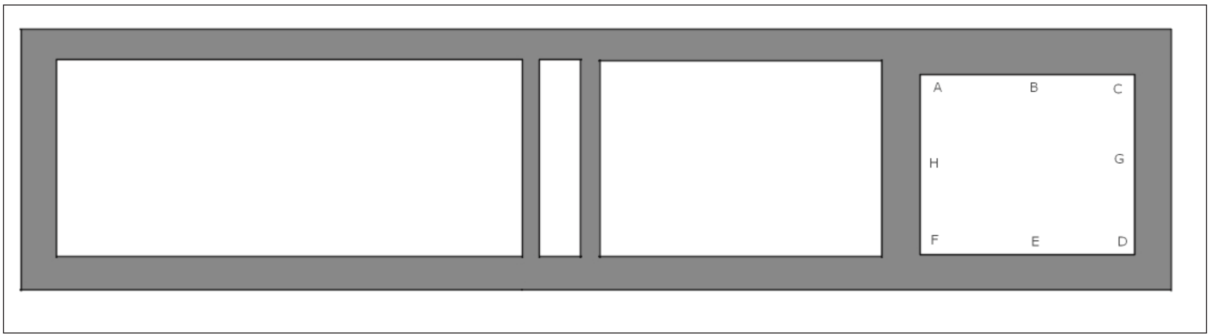


Figure K-11 Distinguished locations for the reinforcement

Roof						
A	Ø	32	-	120		bottom 1
A	Ø	32	-	100		bottom 2
A	Ø	20	-	150		top 1
A	Ø	20	-	150		top 2
A	Ø	25	-	130	- 900	stirrups
B	Ø	40	-	100		top 1
B	Ø	32	-	120		top 2
B	Ø	20	-	150		bottom 1
B	Ø	20	-	100		bottom 2
B	Ø	16	-	200	- 900	stirrups
C	Ø	32	-	120		bottom 1
C	Ø	32	-	100		bottom 2
C	Ø	20	-	100		top 1
C	Ø	20	-	150		top 2
C	Ø	32	-	100	- 900	stirrups

Floor						
D	Ø	40	-	140		top 1
D	Ø	32	-	130		top 2
D	Ø	20	-	200		bottom 1
D	Ø	20	-	200		bottom 2
D	Ø	32	-	180	- 900	stirrups
E	Ø	40	-	120		bottom 1
E	Ø	32	-	120		bottom 2
E	Ø	20	-	200		top 1
E	Ø	20	-	200		top 2
E	Ø	25	-	100	- 900	stirrups
F	Ø	32	-	100		top 1
F	Ø	32	-	100		top 2
F	Ø	20	-	200		bottom 1
F	Ø	20	-	200		bottom 2
F	Ø	32	-	200	- 900	stirrups

Outer wall						
C	Ø	40	-	130		inside 1
C	Ø	32	-	100		inside 2
C	Ø	20	-	150		outside 1
C	Ø	20	-	150		outside 2
C	Ø	25	-	120	- 900	stirrups
G	Ø	40	-	100		outside 1
G	Ø	32	-	200		outside 2
G	Ø	20	-	200		inside 1
G	Ø	20	-	200		inside 2
G	Ø	16	-	200	- 900	stirrups
D	Ø	40	-	100		inside 1
D	Ø	32	-	150		inside 2
D	Ø	20	-	150		outside 1
D	Ø	20	-	200		outside 2
D	Ø	25	-	110	- 900	stirrups

Intermediate wall						
A	Ø	40	-	120		inside 1
A	Ø	32	-	100		inside 2
A	Ø	20	-	100		outside 1
A	Ø	20	-	100		outside 2
A	Ø	40	-	100	- 900	stirrups
H	Ø	40	-	120		outside 1
H	Ø	32	-	120		outside 2
H	Ø	12	-	250		inside 1
H	Ø	16	-	200	- 900	stirrups
F	Ø	40	-	100		inside 1
F	Ø	32	-	100		inside 2
F	Ø	20	-	200		outside 1
F	Ø	32	-	100	- 900	stirrups

Table K-4 Required reinforcement special tube

The ratio between the required amount of reinforcement and concrete is a suitable indication for the constructability of the the alternative. For the considered locations, this ratio is listed in the table below.

Location	Element	[kg/m³]
A	Roof	143
B	Roof	140
C	Roof	198
D	Floor	185
E	Floor	204
F	Floor	184
C	Outer wall	203
G	Outer wall	144
D	Outer wall	206
A	Intermediate wall	276
H	Intermediate wall	120
F	Intermediate wall	210

Table K-5 Ratios reinforcement- concrete for the special tube

From these calculations it can be concluded that relatively large amounts of shear reinforcement are required locally. It will be not a problem to accommodate sufficient capacity however. Relatively high ratios are required locally, though the constructability is not considered to be a problem, since for the major part of the cross-section regular amounts are required.

Conclusion

From these exploring calculations, it can be concluded that applying one separate tube accommodating one lane and an emergency lane can be achieved reasonably. The cross-section could be further optimized, though the order of magnitude of the several dimensions is quite representative, considering the results of the calculations.

K.2 2 special tubes

In case it is desired to apply 2 separate tubes, the cross-section can be designed a little more efficiently. The reason for this is that the stability is not an issue since the cross-section is almost symmetric. Therefore no stabilizing ballast is required during transport and therefore the draught will be smaller. An example of a possible solution is presented in the figures below.

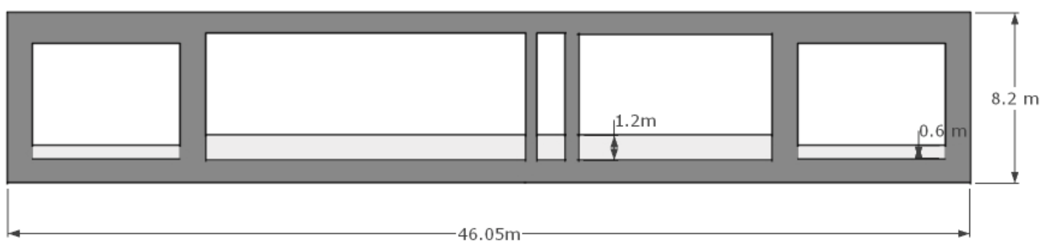


Figure K-12 Lay-out 2 special tubes

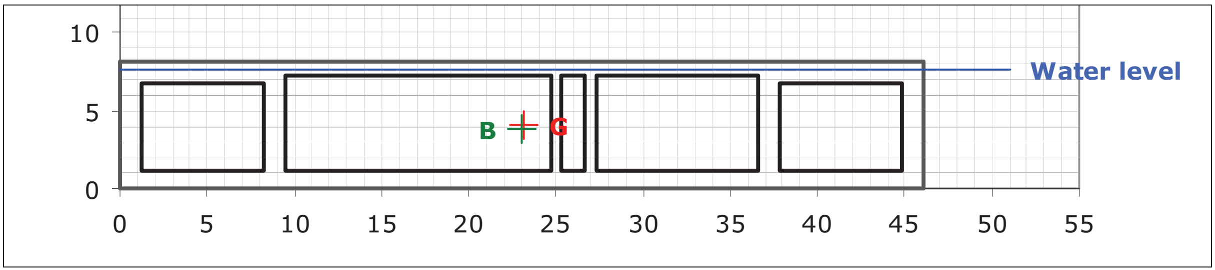


Figure K-13 Stability check during transport

The results of the stability calculation for the application of two special tubes are listed in the table below.

Effective height	5	[m]
Thickness roof slab special tube	1.5	[m]
Thickness floor slab special tube	1.2	[m]
Thickness floor slab	1.1	[m]
Thickness roof slab	0.9	[m]
Ballast height special tube	0.5	[m]
Ballast height regular tube	1.2	[m]
Thickness outer wall 1	1.2	[m]
Width special tube 1	7	[m]
Thickness seperation wall 1	1.3	[m]
Width tube 1	15.25	[m]
Thickness intermediate wal 1	0.6	[m]
Width corridor	1.35	[m]
Thickness intermediate wal 2	0.6	[m]
Width tube 2	9.25	[m]
Thickness seperation wall 2	1.3	[m]
Width special tube 2	7	[m]
Thickness outer wall 2	1.2	[m]
Overall width	46.05	[m]
Overall height	8.2	[m]
Concrete total	140.3	[m²]
Ballast concrete total	39.4	[m²]
Bulk heads + immersion equipement	30	[kN/m]
Weight	3539	[kN]
Maximal buoyancy force	3776	[kN]
Weight/ max buoyancy force	0.94	[-]
Freeboard	0.52	[m]
Safety in final situation	1.12	[-]

Table K6 Results of the stability calculation

The structural aspects and required amounts of reinforcement are similar to the results of 1 special tube and for the walls evens slightly more favourable as a result of the reduced height.

K.3 2 lanes

The application of a special tube, accommodating two lanes, separated by a barrier is investigated. The required thicknesses are estimated from exploring calculations and with help of the mass spring model. With the stability calculations the following solution is obtained.

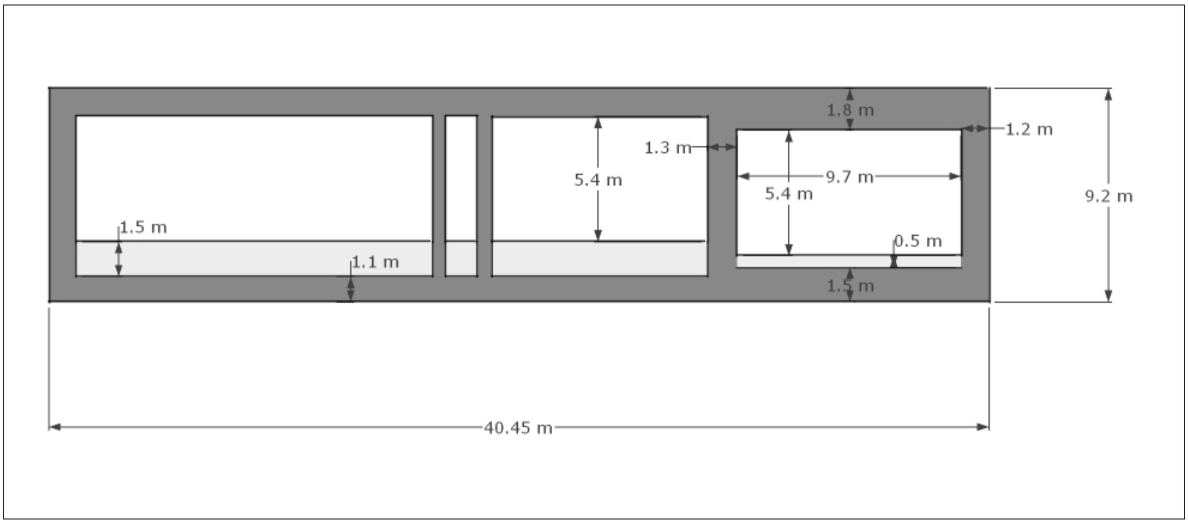


Figure K-14 Lay- out two lanes

Again, water ballast is required in order to provide sufficient stability during transport.

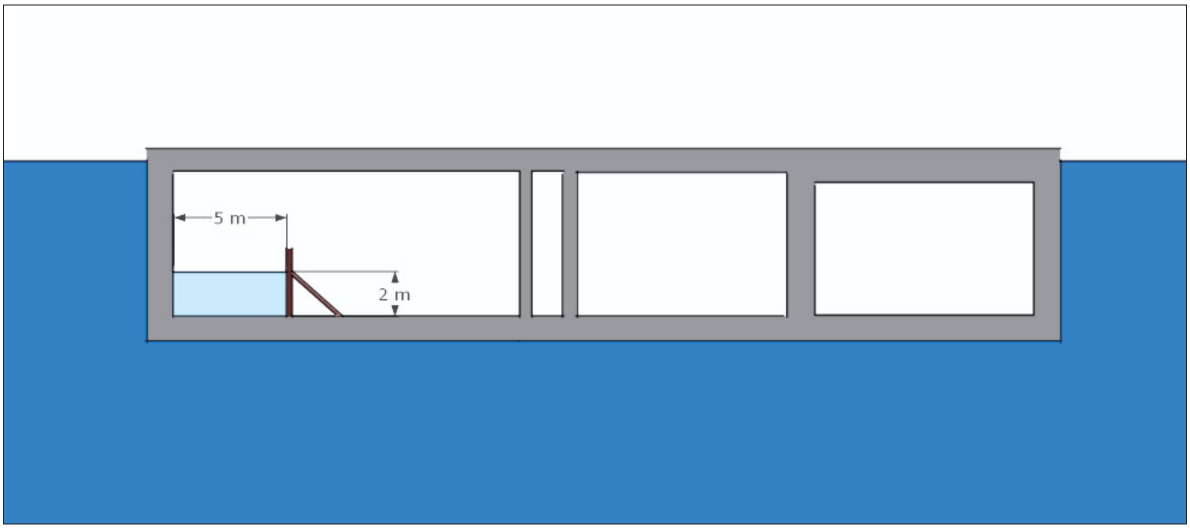


Figure K-15 Application of water ballast

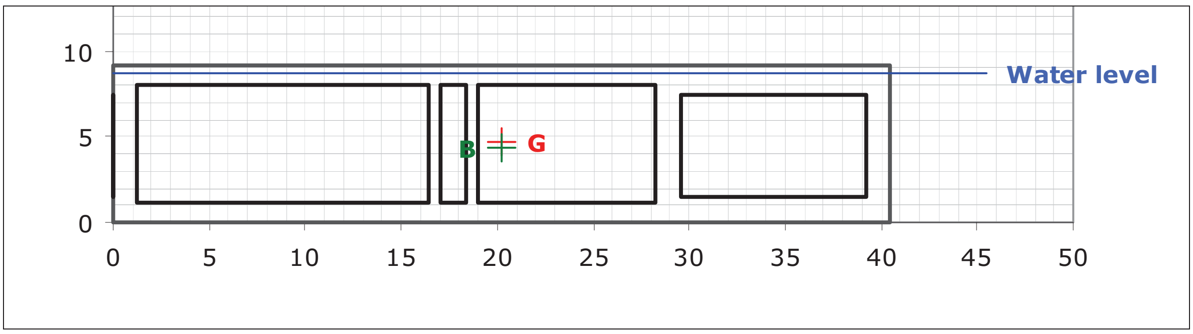


Figure K-16 Stability check during transport

The results of the stability calculation are listed in the table below.

Effective height	5.4	[m]
Thickness roof slab special tube	1.8	[m]
Thickness floor slab special tube	1.5	[m]
Thickness floor slab	1.1	[m]
Thickness roof slab	1.2	[m]
Ballast height special tube	0.5	[m]
Ballast height regular tube	1.5	[m]
Thickness outer wall 1	1.2	[m]
Width tube 1	15.25	[m]
Thickness intermediate wal 1	0.6	[m]
Width corridor	1.35	[m]
Thickness intermediate wal 2	0.6	[m]
Width tube 2	9.25	[m]
Thickness seperation wall 2	1.3	[m]
Width special tube 2	9.7	[m]
Thickness outer wall 2	1.2	[m]
Overall width	40.45	[m]
Overall height	9.2	[m]
Concrete total	136.5	[m²]
Water ballast transport	10	[m²]
Ballast concrete final	43.6	[m²]
Bulk heads + immersion equipement	30	[kN/m]
Weight	3516	[kN]
Maximum buoyancy force	3721	[kN]
weight/ max buoyancy force	0.94	[-]
freeboard	0.51	[m]
Safety in final situation	1.14	[-]

Table K-7 Results of the stability calculation for 2 lanes

For the dynamic analysis with Plaxis, the following schematisation of the special tube, accommodating two lanes was used.

Property	Roof	Floor	Outer wall	Int. wall	Unit
EA	5.58 E7	4.65 E7	3.72 E7	4.03 E7	[kN/m]
EI	1.51 E7	8.719 E6	4.46 E6	5.67 E6	[kN/m²/m]
d	1.8	1.5	1.2	1.3	[m]
w	25	25	25	25	[kN/m/m]
Mp	1.73 E4	1.91 E4	7545	8890	[kNm/m]
Np	1.17 E4	9787	7830	8482	[kN/m]

Table K-8 Properties of the special tube as used in the calculations with Plaxis

The displacements for the point indicated in figure K-17 are presented graphically in figure K-18.

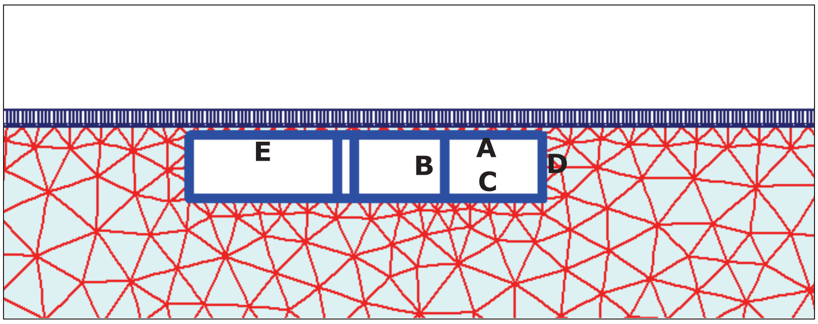


Figure K-17 Location of the considered points

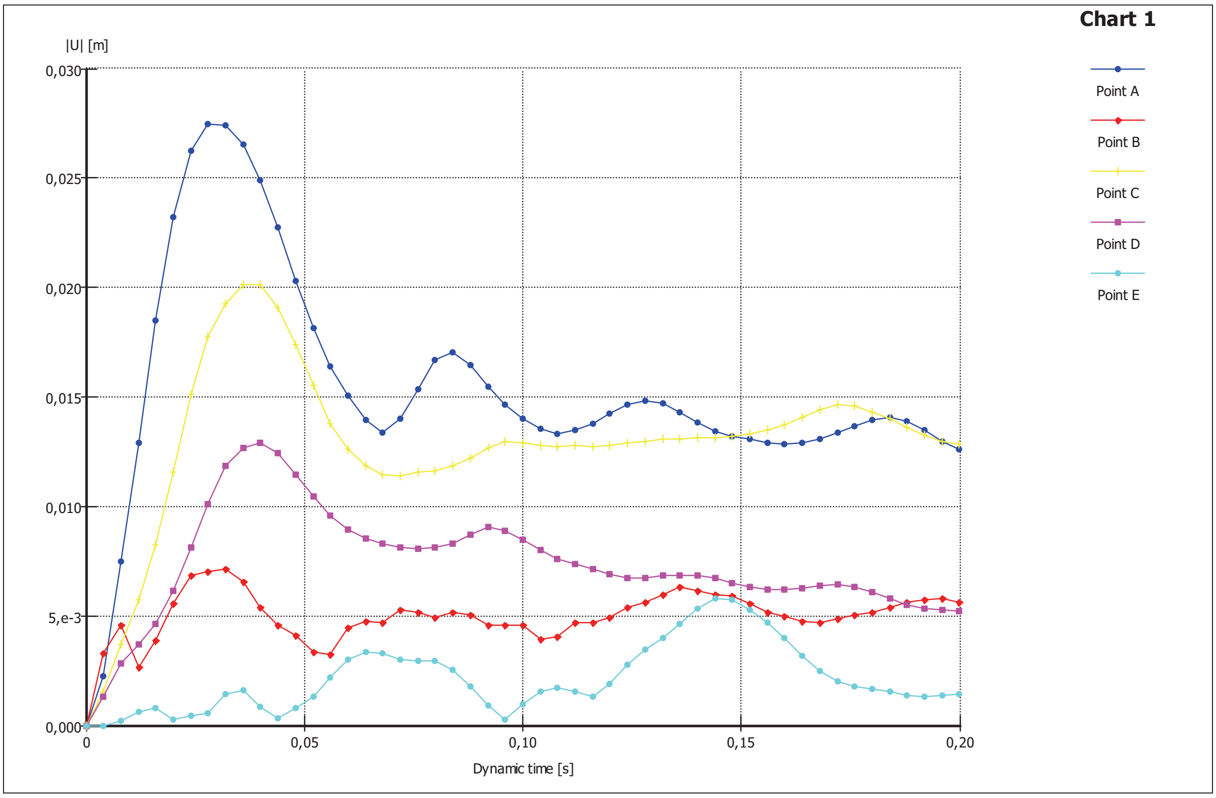


Figure K-18 Displacements as a function of time

The envelope diagram for the occurring bending moments in the cross-section is presented in the figure below.

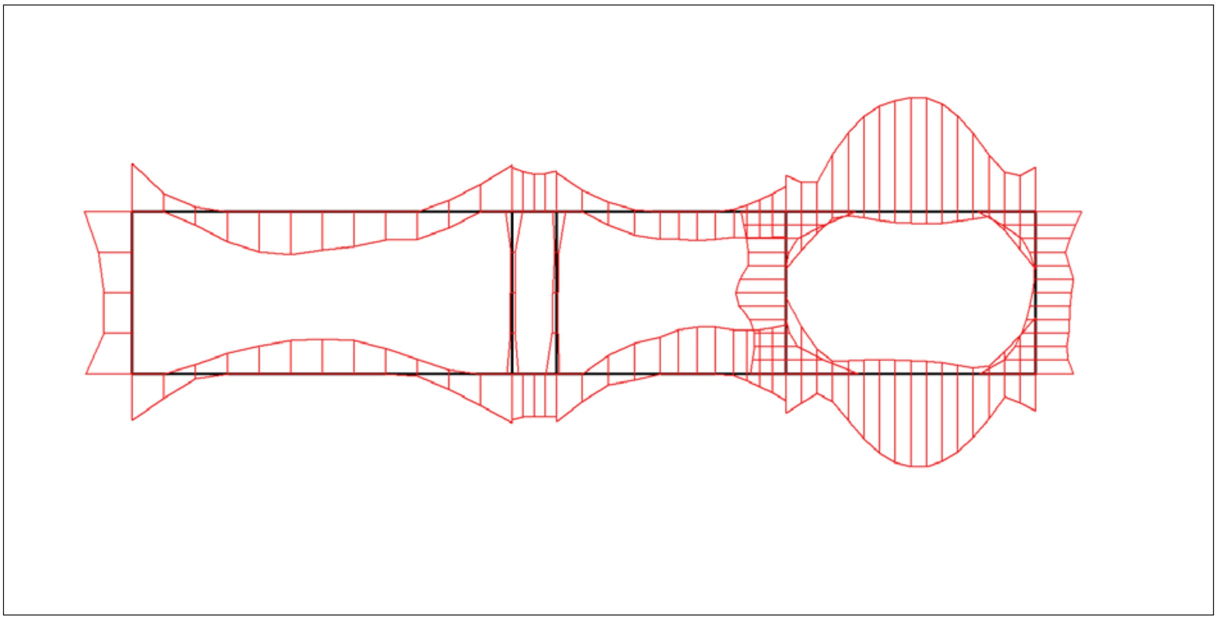
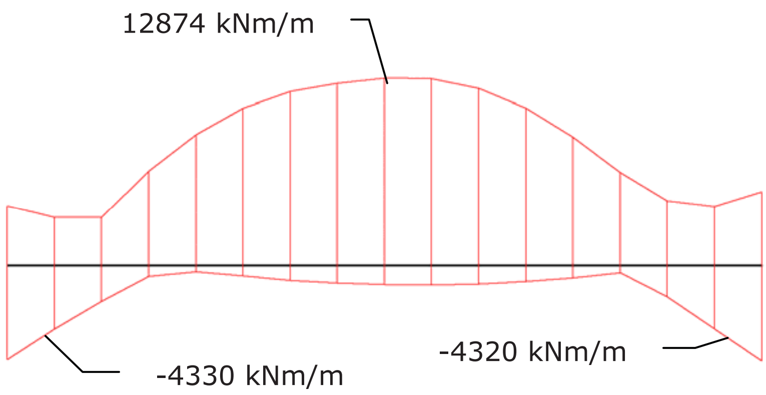
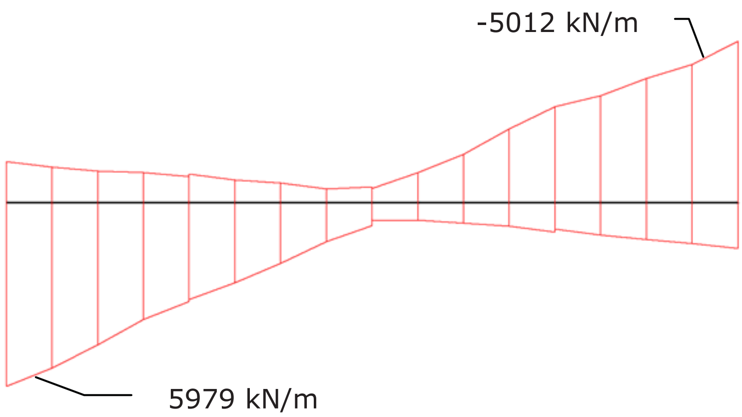


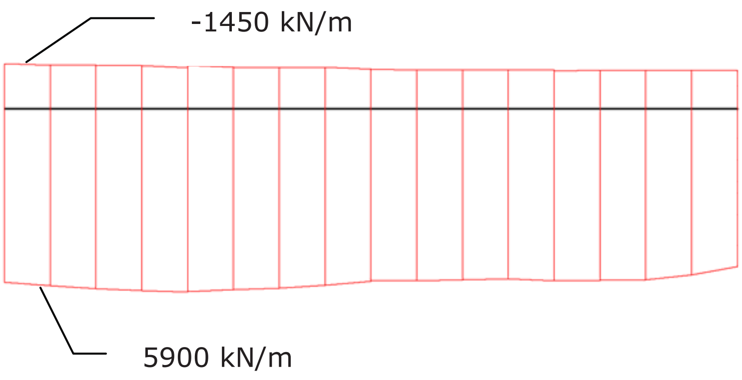
Figure K-19 Envelope diagram for the bending moments

Since the special tube is of particular interest, the envelope diagram, shear force diagram an axial forces diagram is given for each element.

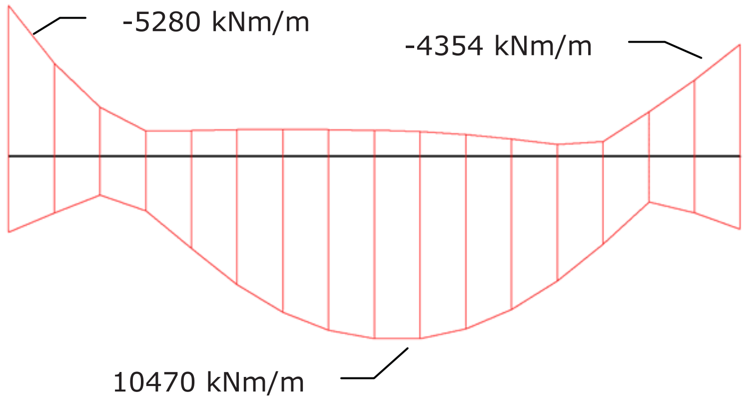
Roof special tube

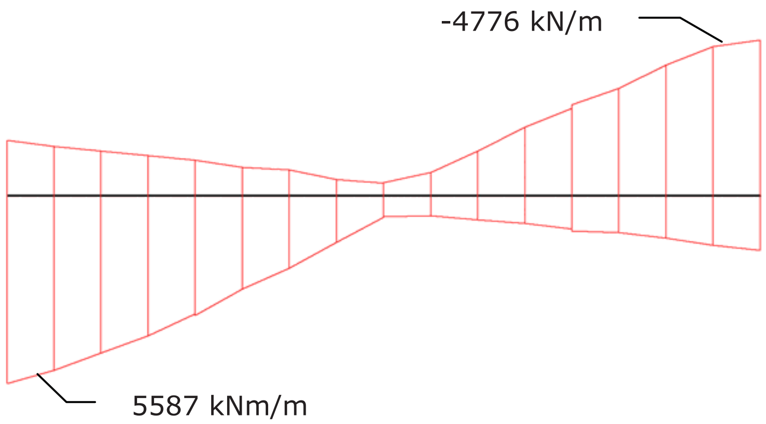


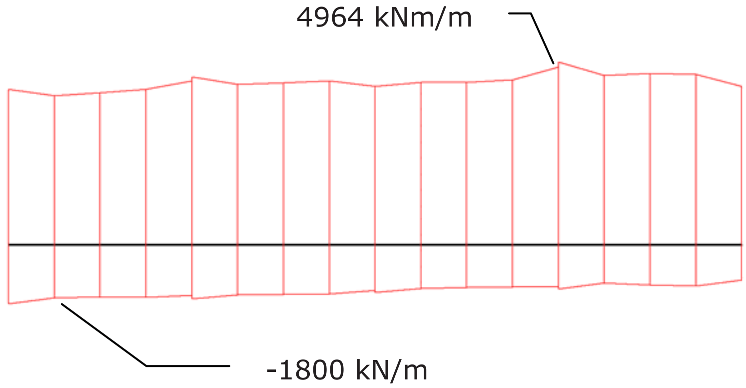




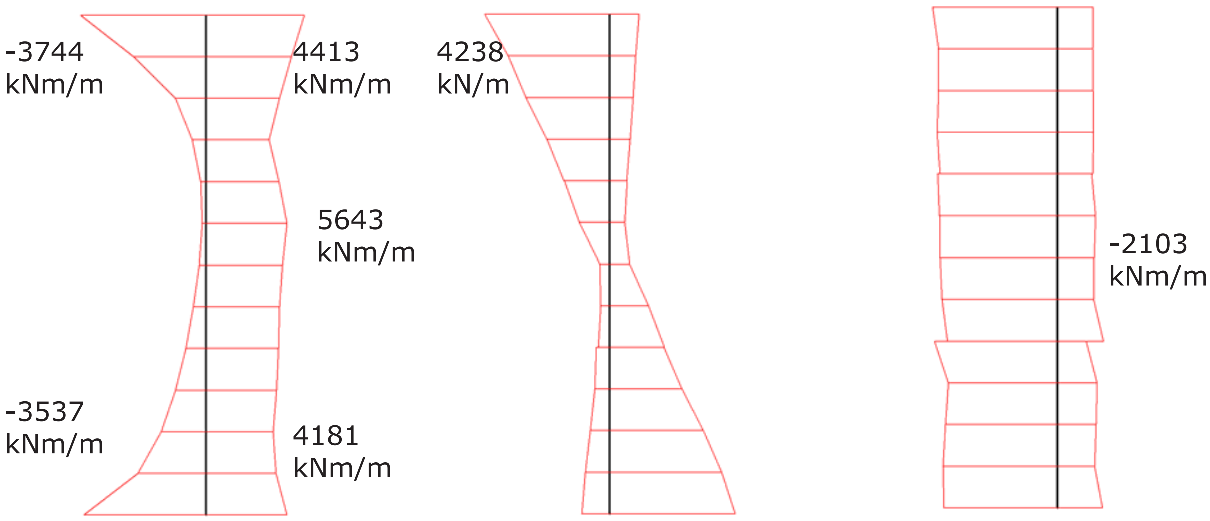
Floor special tube



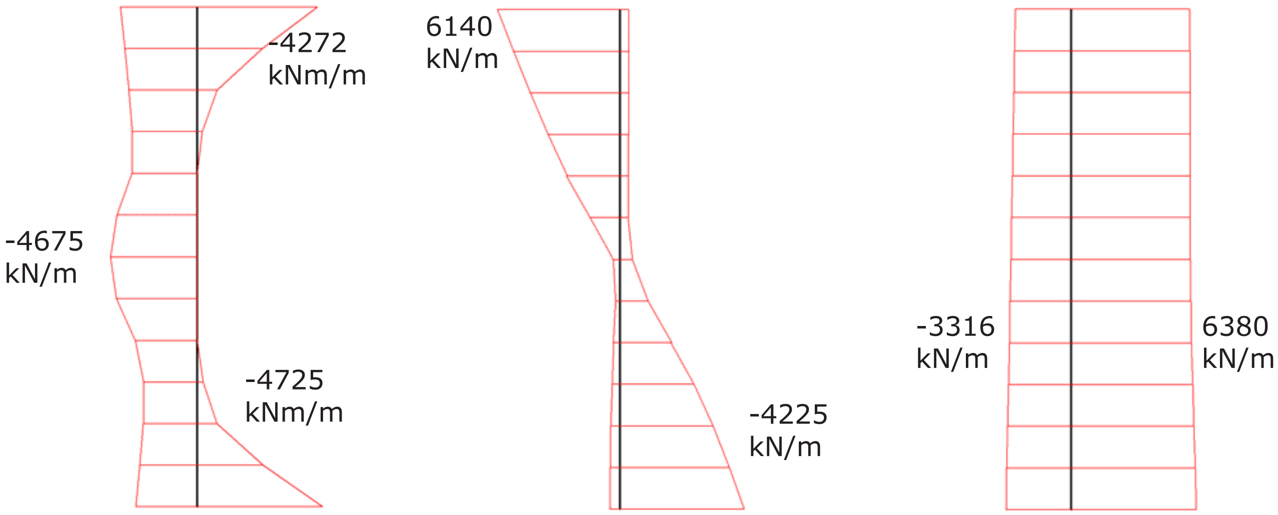




Outer wall special tube



Intermediate wall special tube



It can be concluded that the plastic bending moments are not exceeded as a result of the representative explosion load for the considered configuration.