

Van Mourik Broekmanweg 6
P.O. Box 49
2600 AA Delft
The Netherlands

www.tno.nl

T +31 15 276 30 00
F +31 15 276 30 18

TNO report

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**Effect of explosions in tunnels
Preliminary assessment of the structural response**

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Author(s) dr.ir. A.H.J.M. Vervuurt
dr.ir. F.M.B. Galanti
ir A.J. Wubs
ir. A.C. van den Berg (D&V)

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1 Introduction

1.1 Background of the project

In the Netherlands the available land is used more and more intensively. Main corridors of transport (roads and railroads) are part of the urban area. In order to avoid the negative influences of the corridors of transport (noise, pollution, barriers for local transport) many main corridors of transport will be built in tunnels. The responsible authorities have to decide whether dangerous goods may be transported through these tunnels. First, their attention focuses on the safety of human beings in the tunnel. However, also the integrity of the structure and the economic consequences of an accident must be considered. For the last aspect, knowledge of the loading mechanism and the structural response is required.

Nowadays the goods which are sensitive for explosion are transported along alternative routes that exclude tunnels. These are mostly secondary roads. The transport along these alternative roads has many disadvantages, such as the safety along the route, the air- and noise pollution along the road and the higher transport costs. Therefore, it is preferred to permit the transport of dangerous goods through tunnels. In case of multiple use of space this leads to the question what are the possible consequences and risks for buildings of structures above the tunnel.

In the Delft Cluster work package “Bijzondere Belastingen” (CT01.21) the consequences of an accident with a transport of explosion hazardous goods are considered: BLEVE¹ and gas explosion. These phenomena have a low probability of occurrence, but might have immense consequences. Therefore, a deterministic consideration is not possible.

The results of the work package must facilitate the quantitative risk analysis of the phenomena, which supports the authorities in their decision of allowing transport of dangerous goods through tunnels or not. The work package focus is on the mechanical description of the loading and the response. However, it requires an interdisciplinary approach, which integrates knowledge of risk analysis, explosion and evaporation of liquefied gases, structural dynamics and soil dynamics.

1.2 Project description

The work package contains two main stream research lines:

1. Loading due to BLEVE and gas explosion. The BLEVE research is mainly executed in a PhD project at Delft university of Technology. This part focuses on an improved understanding and modeling of the BLEVE phenomenon. TNO Defense and Safety will participate in this research line by introduction of practical

¹ BLEVE (Boiling Liquid Expanding Vapor Explosion) is the phenomenon of an extremely fast evaporation of liquefied gas that occurs after the containing vessel has failed. Blast waves are generated which are comparable to the blast of an explosion. Bursting vessels with pressurized gas will also be addressed in the current study.

mechanical modeling of the vessel behavior and creation of a practical engineering model for BLEVE load prediction, based on the results of a PhD-study.

2. Dynamic Response of the structure-soil system under BLEVE and gas explosion loading. Here TNO Built Environment and Geosciences concentrates on the structural part of the problem, whereas GeoDelft and Delft University of Technology will take care of the soil response. TNO Defense and Safety will provide data on appropriate loads for realistic cases.

A full description of the project plan is given in [1]

1.3 Focus

The current document reports on the first part of research line 2 (structural response). For studying the soil response due to large deformations (phase 2 of research line 2), finite element simulations have been carried out for studying the effect of high rate loadings (such as for example a gas explosion and a BLEVE) on the expected wall and roof deformations. The results of these analyses are presented in the current report.

The simulations are based on assumptions that provide for an initial guess with respect to the expected response. In the final phase (phase 3 of research line 2, see [1]) more detailed analyses are foreseen. For studying the expected structural response, relevant loading conditions and material properties are provided by TNO D&V (current state-of-the-art).

2 Models

2.1 Introduction

For estimating the expected response of a tunnel structure due to explosion loading conditions, different tunnels and load scenarios are considered. In section 2.2 the tunnel geometries, loading scenarios and the adopted finite element model are briefly outlined. In section 2.3 to 2.5 the geometry of the tunnels considered are explained in detail.

For simplicity of the model, soil on top of the tunnel and on both sides of the tunnel, has been taken into account as a load only. Neither the effect of the mass on top of the tunnel and next to the tunnel is modeled, nor the effect of the embedding stiffness of the soil. The effect of the mass of the tunnel itself is taken into account in the analyses. For the expected deformations in time this is assumed to be a conservative assumption, at least for the deflections of the roof in the first response. For the rebound the roof deflections will be less. In Annex H the effect of the surrounding soil is studied briefly with a membrane model.

2.2 Analyses performed

Tunnel geometries

Three tunnel geometries have been analyzed. These tunnels are explained in detail in section 2.3, 2.4 and 2.5. The analyses are a continuation of the analyses reported in [3] and [7]. In the reports the behavior of the land tunnel “Leidsche Rijn” (situated in highway A2 near Utrecht, The Netherlands) has been analyzed. For that reason this tunnel is explained only briefly in section 2.5. Moreover, the Caland tunnel (section 2.3) and the Drecht tunnel (section 2.4) are analyzed.

Loading conditions

In total three load conditions for the explosion are considered (load scenario 1, 2 and 3 respectively). Load scenario 1 is based on the assumptions as given in [2]-[5]. In these analyses a BLEVE load is applied, based on:

- 1) A distance of 25 m to the explosion source
- 2) An exponential decrease of the load
- 3) An impulse depending on the geometry of the tunnel

Moreover the actually applied load depends on the internal dimensions of the tunnel section. For that reason the load is different for all three tunnels considered. The loads are explained in the corresponding sections.

For the Leidsche Rijn tunnel (section 2.5) two additional load scenarios have been considered [7]. Load scenario 2 applies to a short pulse load in the impulsive area (rather high peak load and a high decreasing time/load rate), whereas load scenario 3 applies to a load in the quasi static area (rather low load and a low time/load rate).

Finite element model

All cases have been modeled using the finite element package DIANA. A beam model has been used, using plane stress beams (CL9BE). For taking into account non

linearity's in some of the analyses, physical non linear material behavior is assumed. Moreover, in these analyses, the reinforcement is modeled explicitly (as embedded reinforcement). Because a beam model has been used, shear reinforcement is not taken into account. Geometrical non linearity's have not been taken into account.

As mentioned earlier, soil on top of the tunnel and on both sides of the tunnel, has not been taken into account, except for the loading. For the expected deformations in time this is assumed to be a conservative assumption. In Annex H the effect of the surrounding soil is studied briefly using a membrane model.

Analyses performed

Six analyses have been performed (id 01 to id 06, see Table 1). The Caland tunnel is analyzed linear elastically (id 01) as well as taking into account the non-linear material behavior (id 02). The Drecht tunnel (id 03) is only analyzed linear elastically. The reason that no non linear behavior has been considered is that the reinforcement data was not fully available.

For the Leidsche Rijn tunnel three non linear cases are considered, based on the three loading regimes (id 04, id 05 and id06 for load scenario 1, 2 and 3 respectively).

Table 1 Performed analyses

Name	analyses number	physically linear	physically non linear	load scenario		
				1	2	3
Caland tunnel	id 01					
	id 02					
Drecht tunnel	id 03					
Leidsche Rijn tunnel	id 04					
	id 05					
	id 06					

Load and time steps

The load is applied stepwise.

- In step 1 to 10 the dead weight is applied (10% of the dead weight per load step).
- In step 10-20 permanent loads are applied (10% of the load per step).
- Thereafter time steps of 10^{-5} sec are applied. In all analyses 10000 time steps are applied, thereby analyzing 0.1 sec of the response of the tunnel.

The results indicate that the analyses time (0.1 sec) is enough time for studying the initial response of the structure. For studying the effects that play a role at a longer time span and for studying gradual failure (such as observed in some of the analyses) the time span considered may be too short. In these cases, however, analyzing a larger time span does not give any additional relevant information.

2.3 Caland tunnel

2.3.1 Geometry, dimensions and boundary conditions

An overview of the geometry of the Caland tunnel is given in Figure 1. The finite element model (beam model) of the tunnel is given in Figure 2. In the figures, the

geometry and dimensions are given (spans, thicknesses), as well as the boundary conditions adopted in the FE model. In the circles the group names as adopted in the FE model are given.

It can be seen from the figure that the bottom slab of the tunnel is not modeled, thereby assuming that the slab of the tunnel is infinitely stiff. Considering the loads on the floor of the tunnel, this is an acceptable assumption. For simplicity of the model, also no foundations have been modeled.

Moreover, it can be seen from the figure that near the mid walls (wall 2 and 3), the beams follow the actual shape of the tunnel and an altering height of the beams is adopted over a length of 3 m from the walls.

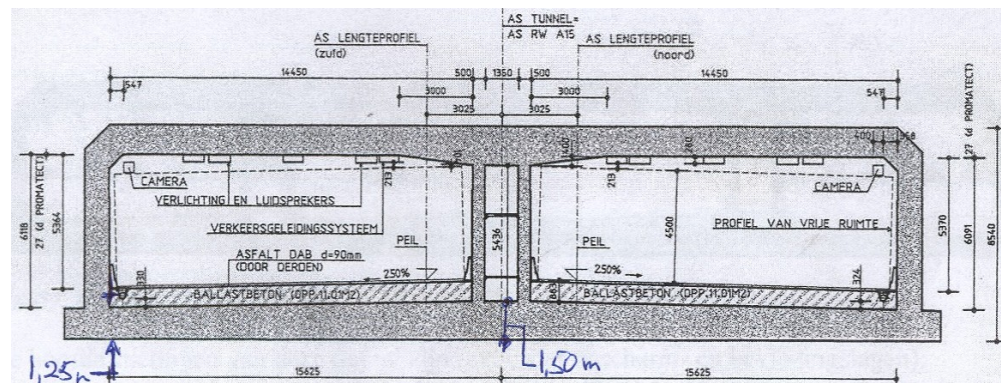


Figure 1 Geometry and dimension of the Caland tunnel

From [7] it appeared that schematizing the system lines of the tunnel may lead to an overestimation of the deflections and underestimation of the frequency predicted when compared to schematizing the inner span of the tunnel walls and the roof. For that reason the inner span is modeled for both the Caland tunnel and the Drecht tunnel (section 2.4). For the Leidsche Rijn tunnel (section 2.5) an existing model is used in which the system lines have been taken into account rather than the inner span measures.

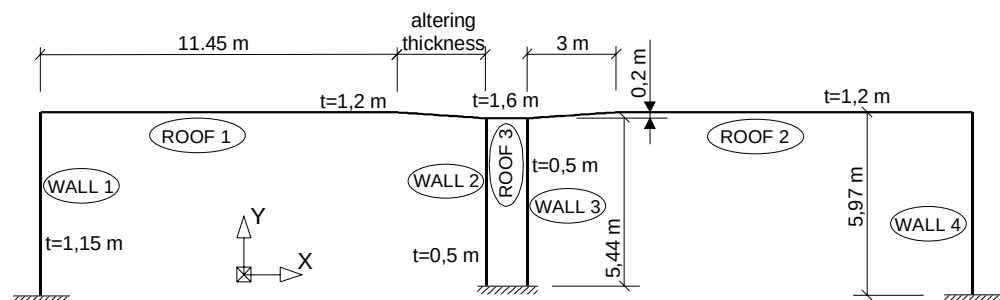


Figure 2 Finite element model of the Caland tunnel

The reinforcement is given in Figure 3. In the figure the group names of the different reinforcement sections are given, as well as the location of the rebars. The geometry (cross sectional areas, length) and the concrete cover to the rebars are given in Table 2. Shear reinforcement is not taken into account. For calculating shear failure, the calculated shear force may be verified to rules present in the codes. Because of time available and focus of this part of the project, shear failure is not considered. An example for analyzing shear failure is elucidated in [7].

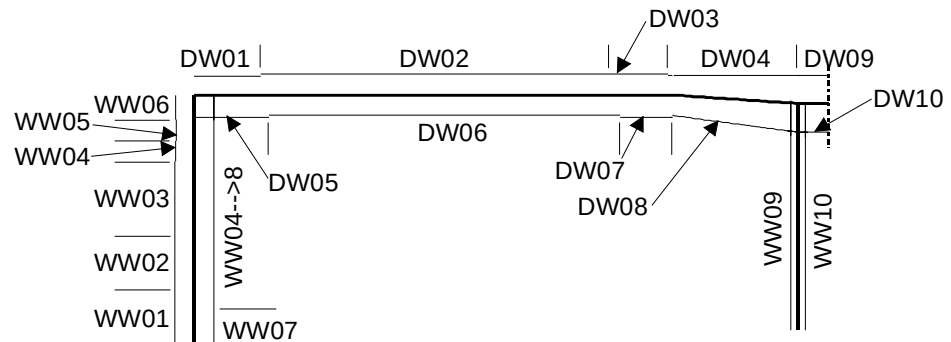


Figure 3 Group names of the reinforcement in the FE model. The reinforcement is detailed in Table 2

Table 2 Reinforcement properties: A_s represents the cross sectional area and c' is the concrete cover from the gravity centre of the rebars.

Name	A_s (mm ² /m)	c' (mm)	length (m)
Walls			
WW01	1340 ^{*)}	62	full wall height
WW02	2094	81	full wall height
WW03	1047	91	1.3
WW04	2094	91	remaining length
WW05	3727	123	1.3
WW06	4774	113	1.3
WW07	7454	123	remaining length
WW08	8500	134	0.5
WW09	11180	153	0.5
WW10	8500	134	0.59
Roof			
DW01	7454	143	1.6
DW02	2094	96	8.35
DW03	5360	102	1.4
DW04	13400	131	3.1
DW05	2094	65	1.8
DW06	7454	112	8.4
DW07	2094	65	1.25
DW08	2094	65	3.0
Rescue tube			
DW09	13400	65	full roof width
DW10	2094	131	full roof width

^{*)} excluding crossing reinforcement from the floor of the tunnel

2.3.2 Material properties

For modeling the non linear behavior, physical non linear material properties are adopted. For this purpose a total strain crack model is used for modeling concrete cracking (Figure 4a). For concrete under compression and the steel reinforcement a yield criterion is used (Figure 4b and Figure 5).

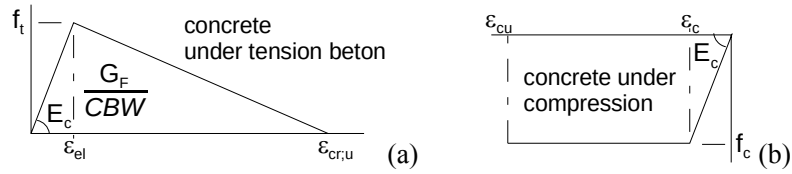


Figure 4 Concrete material properties: stress strain relation in tension (a) and compression (b)

The adopted material properties for the analyses regarding the Caland tunnel are given in Table 3. In the table all relevant properties are summarized. An example of the DIANA syntax is given in Annex A.1.

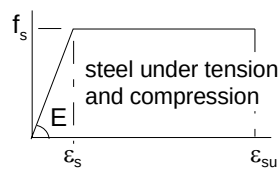


Figure 5 Steel material properties: stress strain relation

Table 3 Material properties applied in the analyses of the Caland tunnel.

Property	Value
concrete strength class	B 35 (35 MPa characteristic strength)
compressive strength	$f_c = 0.85 \cdot f_{ck} / \gamma_m = 0.85 \cdot 35 / 1.0 = 29.8 \text{ N/mm}^2$
compressive failure strain	$\epsilon_{cu} = 3.5 \text{‰}$
tensile strength	$f_t = 2.33 \text{ N/mm}^2$ (FEM)
modulus of elasticity	$E_c = 31.000 \text{ N/mm}^2$
contraction coefficient	$\nu = 0.15$
mass	$\rho = 2500 \text{ kg/m}^3$
fracture energy	$G_F = 100.0 \text{ N/m}$
Crack Band Width (CBW)	0.062 mm (effective length of 1 integration point)

reinforcement steel	FeB 500 HWL
strength	$f_s = f_{srep} / \gamma_m = 500 / 1.0 = 500 \text{ N/mm}^2$
modulus of elasticity	$E_s = 200,000 \text{ N/mm}^2$
yield strain	$\epsilon_s = 500 / 200,000 = 2.5 \text{‰}$
failure strain	$\epsilon_{su} = 3.25 \text{‰}$
contraction coefficient	$\nu = 0.3$
mass	$\rho = 7850 \text{ kg/m}^3$

2.3.3 Loading conditions

The permanent loads applied in the model are schematically shown in Figure 6. The loads are quantified in Table 4. Moreover, in Figure 6 the explosion load is shown schematically. As can be seen from Figure 6 a constant load is applied on the roofs of the tunnel, whereas on the walls a linearly increasing load is assumed. It is noticed that all permanent loads on the tunnel are massless thereby not taking into account the effect

of the mass on the response on the structure and generally leading to a larger first amplitude of the deflection.

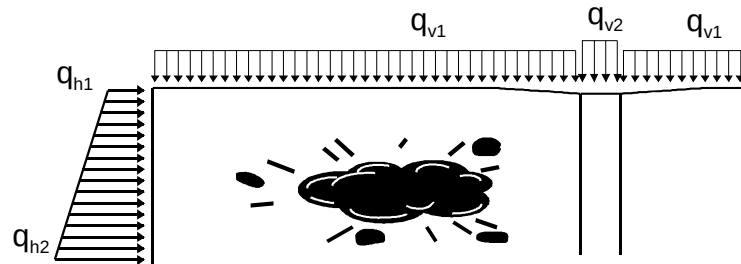


Figure 6 Permanent loads on the tunnel, including the position of the explosion load

Because on the one hand the inner span measures are taken into account rather than the system spans of the tunnel and, on the other hand, the total loads are assumed present, the load on the roof in the middle tube is higher compared to the load on the roof of the main tubes ($q_{v2} > q_{v1}$).

Table 4 Distributed loads on the walls and the roof of the Caland tunnel (according to Figure 6).

Description		load (kN/m ²)
Roof (system line)	q_{v1}	187.3
	q_{v2}	234.0
Wall (system line)	q_{h1}	186.1
	q_{h2}	277.5

The applied explosion load is given in Figure 7. In the figure it is shown that the load increases to 5.13 bar, where after a exponentially decreasing load is assumed. At $t=160$ ms the load is reduced to zero. It is mentioned that the analyses only cover a time span of 100 ms.

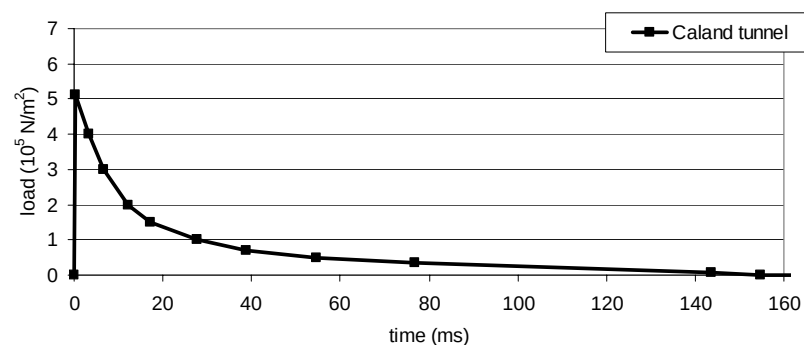


Figure 7 Explosion load characteristics for the Caland tunnel

In the analyses several load cases are considered (Table 5). The load is applied in the subsequent load combinations given in Table 6, according to the following scheme (see also section 2.2):

- Step 1-10: dead weight (combination 3)
- Step 11-20: permanent loading (combination 1)
- Step 21-10021: explosion loading (combination 2) in 10^{-5} sec/step. (total 100 ms)

An example of the DIANA command file adopted is given Annex A.2

Table 5 Load cases for the analyses.

load case	description
1	horizontal loading wall 1
2	horizontal loading wall 2
3	vertical loading (q_{v1})
4	vertical loading (q_{v2})
5	explosion load wall 1 (horizontal)
6	explosion load wall 2 (horizontal)
7	explosion load roof 1 (vertical)
8	dead weight

Table 6 Load combinations applied, multiplication factors

combination	description	load case							
		1	2	3	4	5	6	7	8
1	permanent load	1.0	1.0	1.0	1.0				
2	explosion load					1.0	1.0	1.0	
3	dead weight								1.0

2.4 Drecht tunnel

2.4.1 Geometry, dimensions and boundary conditions

An overview of the geometry of the Drecht tunnel is given in Figure 8. The finite element model (beam model) of the tunnel is given in Figure 9. In Figure 8, the geometry and dimensions are given (spans, thicknesses), as well as the boundary conditions adopted in the FE model.

Contrarily to the model for the Caland tunnel (section 2.3) the altering wall and roof thickness is not taken into account for the model regarding the Drecht tunnel. For simplicity the tubes are modeled as rectangular tubes. The system lines in the model correspond to the dimensions of the inner span measures of the tunnel.

Because the reinforcement data was not fully available, the linear elastic response behavior is analyzed only, and no reinforcement is included in the model (see also section 2.2).

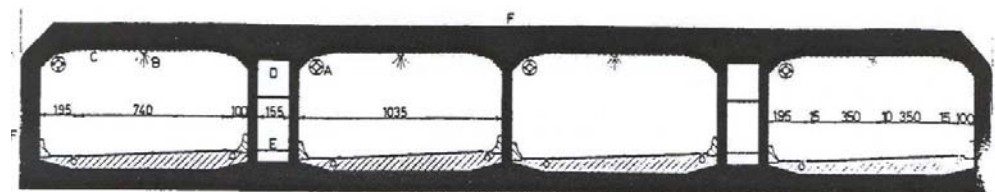


Figure 8 Geometry and dimension of the Drecht tunnel

Table 8 Distributed loads on the walls and the roof of the Drecht tunnel (according to Figure 6)

Description		load (kN/m ²)
Roof (system line)	q _{v1}	133.5
	q _{v2}	158.9
	q _{v3}	125.9
Wall (system line)	q _{h1}	116.4
	q _{h2}	209.5

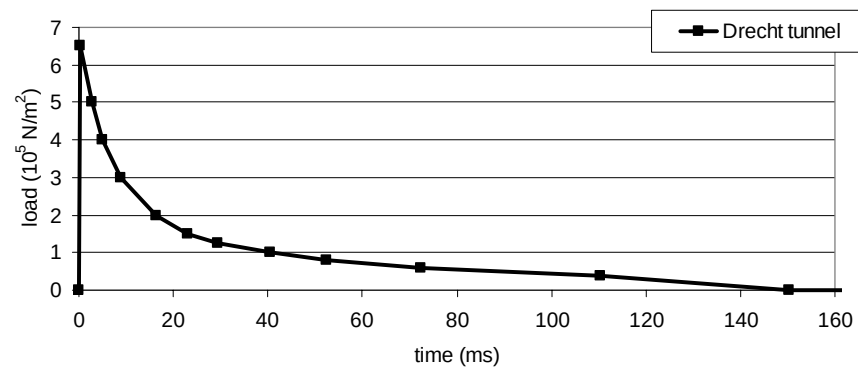


Figure 11 Explosion load characteristics for the Drecht tunnel

2.5 Leidsche Rijn tunnel

2.5.1 Geometry, dimensions and boundary conditions

An overview of the geometry of the Leidsche Rijn tunnel is given in Figure 12. The finite element model (beam model) of the tunnel is given in Figure 13. In the figures, the geometry and dimensions are given (spans, thicknesses), as well as the boundary conditions adopted in the FE model. Moreover the position of the explosion load (in either one of the middle tubes) is given.

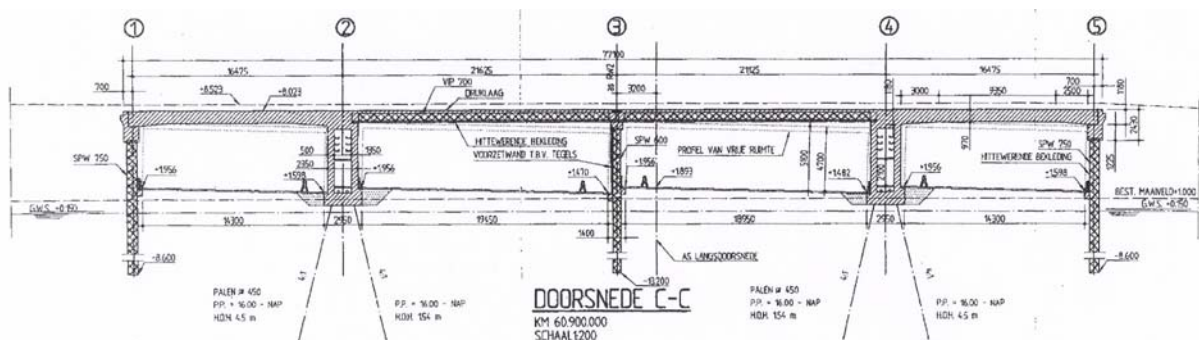


Figure 12 Tunnel Leidsche Rijn

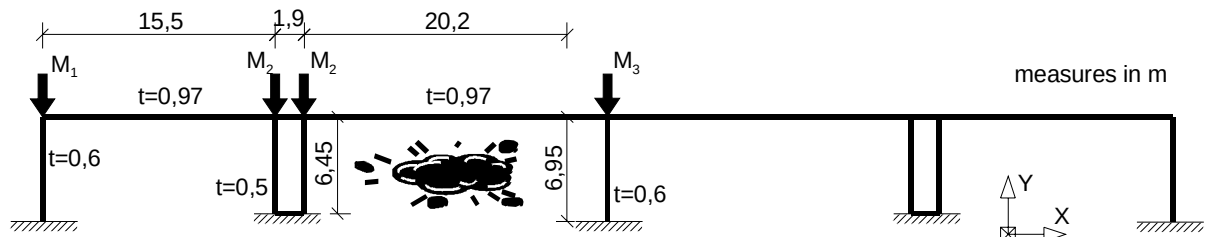


Figure 13 Finite element model of the Leidsche Rijn tunnel

The reinforcement applied in the model is given in Figure 14. The model adopted in this study is fully similar to the model adopted in [7]. For that reason the reader is referred to a full description of the model.

It is noted that the mass loads on top of the walls in the loaded area has a positive effect of the shear and bend capacity of the walls. For that reason an explosion load in the right tunnel section may be more feasible. Because the analyses have been selected similar to the analyses in [7], this effect is not studied.

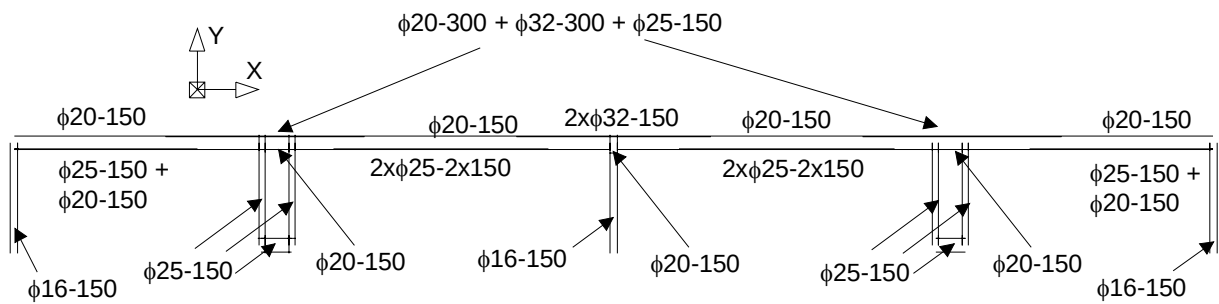


Figure 14 Reinforcement adopted in the finite element model of the Leidsche Rijn tunnel

2.5.2 Material properties

For modeling the non linear behavior, physical non linear material properties are adopted. For this purpose a total strain crack model is used for modeling concrete cracking (Figure 4a). For concrete under compression and the steel reinforcement a yield criterion is used (Figure 4b and Figure 5).

The adopted material properties for the analyses regarding the Leidsche Rijn tunnel are summarized in Table 9. In the table all relevant properties are summarized. An example of the DIANA syntax is given in Annex A.1.

Table 9 Material properties applied in the analyses of the Leidsche Rijn tunnel

Property	Value
concrete strength class	B 45
compressive strength	$f_c = 0.85 \cdot f_{ck} / \gamma_m = 0.85 \cdot 45 / 1.0 = 38.3 \text{ N/mm}^2$
compressive failure strain	$\epsilon_{cu} = 3.5 \cdot 10^{-3}$
tensile strength	$f_t = 2.75 \text{ N/mm}^2$

Property	Value
modulus of elasticity	$E'_b = 33,350 \text{ N/mm}^2$
contraction coefficient	$\nu = 0.15$
mass	$\rho = 2500 \text{ kg/m}^3$
fracture energy	$G_F = 100.0 \text{ N/m}$
Crack Band Width (CBW)	28 mm (effective length of 1 integrationpunt)

Property	Value
reinforcement steel	FeB 500 HWL
strength	$f_s = f_{srep}/\gamma_m = 500/1.0 = 500 \text{ N/mm}^2$
modulus of elasticity	$E_s = 200,000 \text{ N/mm}^2$
yield strain	$\epsilon_s = 500/200,000 = 2.5 \cdot 10^{-3}$
failure strain	$\epsilon_{su} = 3.25 \%$
contraction coefficient	$\nu = 0.3$
mass	$\rho = 7850 \text{ kg/m}^3$

2.5.3 Loading conditions

The permanent loads applied in the model of the Leidsche Rijn tunnel are fully similar to the analyses reported in [7]. Next to the dead weight, a vertical load on the roof of the tunnel is present, as well as an additional vertical mass load on top of the left walls. The mass loading on top of the walls indicate the weight of buildings on top of the tunnel. Because the mass loadings are modeled as point masses, dynamic effects are taken into account.

Table 10 Loads applied in the model of the Leidsche Rijn tunnel

Description		load
Roof	soil and asphalt	1.06 kN/m^2
	traffic	4.0 kN/m^2
Wall (mass loadings)	M_1	$110 \cdot 10^3 \text{ kg/m}$
	M_2	$100 \cdot 10^3 \text{ kg/m}$
	M_3	$180 \cdot 10^3 \text{ kg/m}$

The position of the explosion load is indicated in Figure 13. It is not feasible assumed that the explosion may occur in the (secondary) outer tubes. The magnitude and time shape of the applied explosion load is given in Figure 15.

Compared to the Caland tunnel and the Drecht tunnel two additional load scenarios are considered for the Leidsche Rijn tunnel. The adopted load scenarios are similar to the load scenarios adopted in [7]. Load scenario 1 (Figure 15a) applies to an initial peak load of 4.35 bar, and an exponentially decreasing load to $p=0$ at about $t=80 \text{ ms}$. Load scenario 2 (Figure 15b) applies to a load in the impulsive area (rather high peak load and a high decreasing time/load rate), whereas load scenario 3 (Figure 15c) applies to a load in the quasi static area (rather low load and a low time/load rate). It is noted that load scenario 2 and 3 are based on extremes and not on specific explosion scenarios.

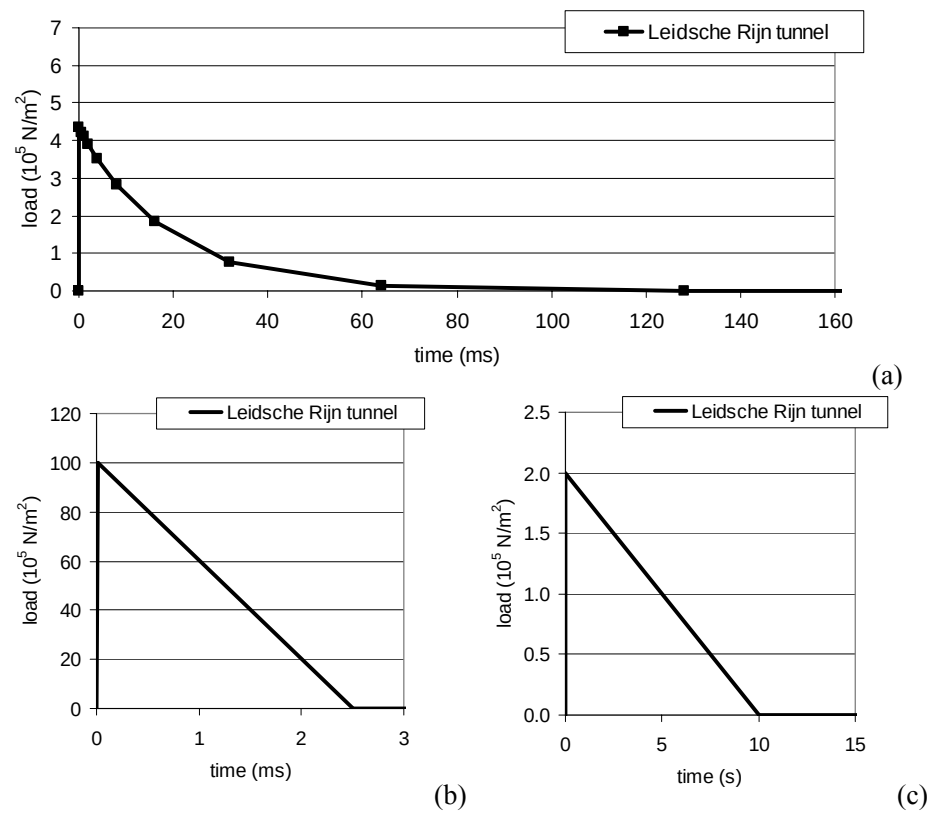


Figure 15 Explosion load characteristics for the Leidsche Rijn tunnel: load scenario 1 (a), 2 (b) and 3 (c)

3 Analyses of the results

3.1 Description of the results

A full overview of the results is given in the annex of the report (annex A-F). In the annex, for each analysis the deflections as well as concrete strains and steel strains are given as a function of the time. Different critical section are considered, see Figure 16.

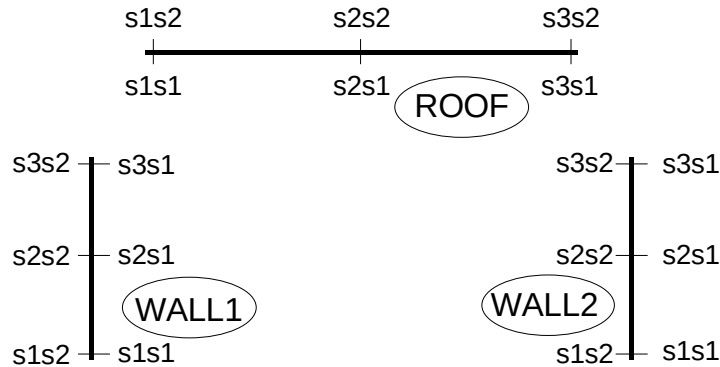


Figure 16 Considered sections

In the following section a short description of the most relevant results is given with respect to the calculated response. Emphasis is given to deformations that may be expected. Moreover a short analysis of the subsequent analyses is given.

3.2 Deflections

3.2.1 Caland tunnel

In the figures below the response behavior of the different analyses is given. In Figure 17 the response for the Caland tunnel is given. Both the linear (solid lines) and the non linear (dashed lines) behavior are given in the figure. The results are summarized in Table 11.

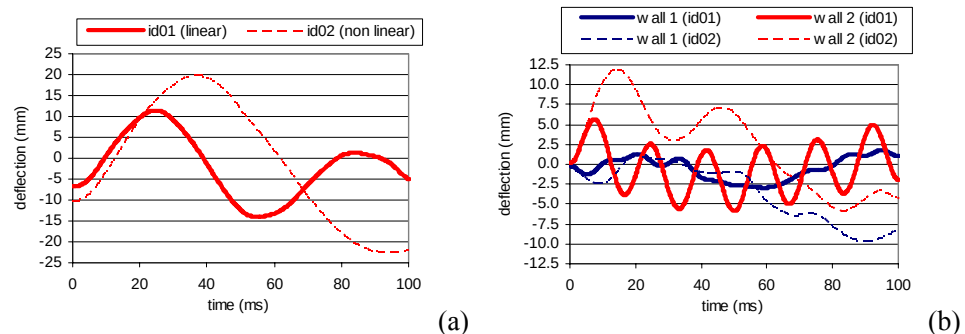


Figure 17 Roof (a) and wall (b) deflections for the Caland tunnel (id 01 and id 02)

From the analyses it is shown that the deflections of both the roof and the walls remains limited to max +/-25 mm. As expected, both the maximum amplitude as well as the frequency strongly depends on the physical material behavior (linear or non linear). The

expected frequency, based on the non linear material behavior, varies between 11 Hz for the roof and 30-45 Hz for the walls. The decrease of the frequency due to the non linear material behavior is roughly a factor 2.

The maximum steel strain calculated from the analyses is about 4.5%. Depending on the element size used in the analyses failure may be expected at $1,5 \cdot \epsilon_{su} - 2 \cdot \epsilon_{su}$ [7]. For $\epsilon_{su}=3,25$ it can be concluded that failure due to bending is not likely for load scenario 1. Considering the main goal of this part of the project, shear failure is not considered in this report, see section 3.3.

Table 11 Expected deformations of the Caland tunnel. The maximum and minimum refer to the absolute values during the analyses whereas the amplitude is determined on the bases of the harmonic response

component	analyses type	maximum (mm)	minimum (mm)	amplitude (mm)	frequency (Hz)
roof	linear	11	-14	12.5	18
	non linear	20	-23	21.5	11
wall 1	linear	1	-3	0.5	80
	non linear	1.1	-10	1.8	45
wall 2	linear	5.5	-5.5	4.7	50
	non linear	12	-6	7.5	30

3.2.2 Drecht tunnel

The results with respect to the Drecht tunnel are summarized in Figure 18 and Table 12. Because the analyses are carried out only taking into account the linear behavior rather harmonic responses are observed. It can be seen clearly that the behavior of both the walls and the roof affect each other. The maximum remains limited to +/- 15 mm for the roof. The amplitude of the outer wall (wall 1) is substantially less compared to wall 2 (similar to the Caland tunnel). This is mainly the result of the larger wall thickness for wall 1.

Because linear elastic material behavior is assumed, failure of the structure is not modeled in the analyses. From the results it appears that cracking is expected. The steel strain governing failure is determined, among others by the ratio of the reinforcement.

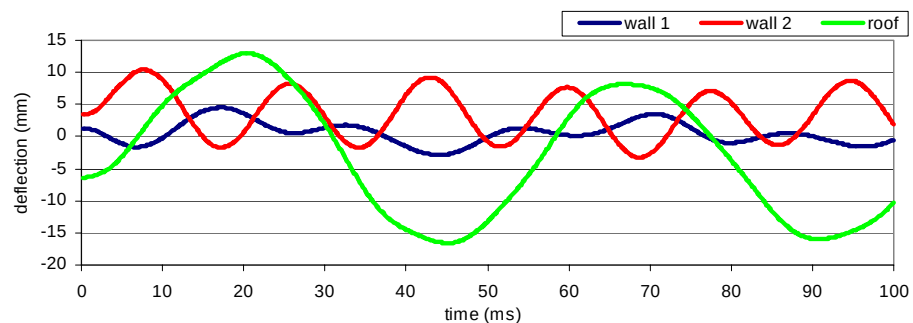


Figure 18 Roof and wall deflections for the Drecht tunnel (id 03)

It appears that the behavior of the Drecht tunnel is rather similar to the Caland tunnel, which could be expected because of the similar dimensions. Provided that the same amount of reinforcement is applied, also similar non linear behavior could be expected.

Table 12 Expected deformations of the Drecht tunnel (linear elastic material behaviour only). The maximum and minimum refer to the absolute values during the analyses whereas the amplitude is determined on the bases of the harmonic response

component	maximum (mm)	minimum (mm)	amplitude (mm)	frequency (Hz)
roof	13	-17	15	22
wall 1	4.5	-2.9	2.5	50
wall 2	10	-3	5.5	55

3.2.3 Leidsche Rijn tunnel

The results with respect to the Leidsche Rijn tunnel are summarized in Figure 19, Figure 20 and Table 12. It can be seen from the results that the displacements of the roof increase monotonically, indicating failure (see also section 3.3), regardless the load scenario. For that reason hardly any results can be extracted for the roof.

For load scenario 2 (Figure 20b), also the walls tend to fail, indicating the expected deformations. For load scenario 3 (Figure 20c), the deformations seem to increase as well, however, more time is needed in the analyses for drawing this conclusion.

From Table 13 it is concluded that the expected frequency is less than 10 Hz for the roof. For the walls frequencies in the order of 30-40 Hz are expected, except for load scenario 2, in which case a frequency of less than 10 Hz is noticed.

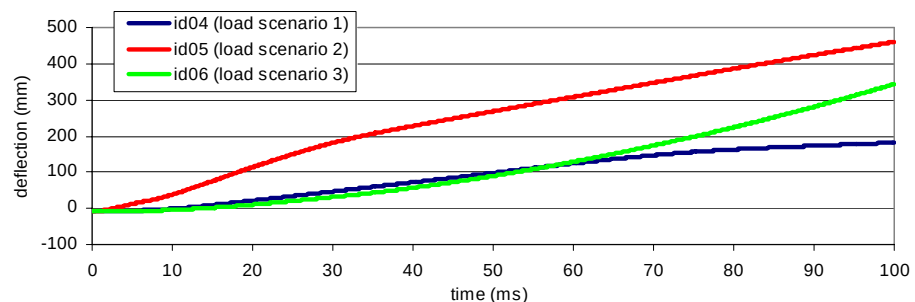


Figure 19 Roof deflections for the Leidsche Rijn tunnel for load scenario 1, 2 and 3 (id 04, id 05 and id 06 respectively)

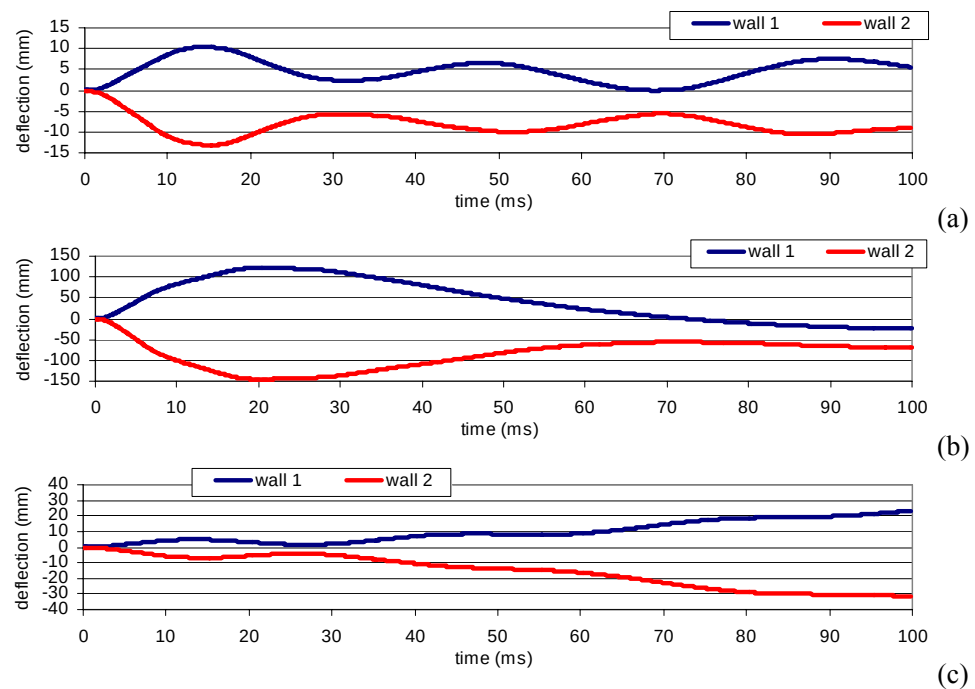


Figure 20 Wall deflections for the Leidsche Rijn tunnel for the three load scenarios: id 04 (a), id 05 (b) and id 06 (c) respectively

Table 13 Expected deformations for three load scenarios applied to the Leidsche Rijn tunnel (non linear material behavior). The maximum and minimum refer to the absolute values during the analyses whereas the amplitude is determined on the bases of the harmonic response

component	maximum (mm)	minimum (mm)	amplitude (mm)	frequency (Hz)
load scenario 1				
roof	> 100	-	> 100	< 10
wall 1	10	0	6	30
wall 2	0	-13	6	30
load scenario 2				
roof	> 100	-	> 100	< 10
wall 1	120	-25	60	< 10
wall 2	0	-150	50	15
load scenario 3				
roof	> 100	-	> 100	< 10
wall 1	> 25	0	3	40
wall 2	0	<30	3	40

3.3 Failure behavior

In the previous sections the deformation behavior of the structural components of three tunnels is considered. In the analyses ideal plastic behavior is assumed for steel failure and concrete failure under compression. In reality however, failure will be induced when the (plastic) strain exceeds a critical value. For concrete under compression crushing will be observed and the steel rebars will fail as well (rupture). At this time the damage will be substantial and at some points the structure will actual fail, or the

damage is that high that the structure cannot be repaired anymore. Of course this also depends on the costs necessary for repairing the structure.

Next to failure of the structure due to exceeding the ultimate tensile/compressive strain, shear failure is not considered in the analyses. Reason for this is that the adopted (beam) models do not support shear failure.

Because the goal of the present study is primarily to obtain an indication of the expected deformation due to an explosion load, no specific attention is paid to possible failure of the structure. In [7], however, a method is given for the failure assessment of the tunnel. Both bending failure and shear failure are addressed in the report.

In the table below an overview is given for the expected damage of the tunnels addressed in the current report. In the table the maximum compressive strain as well as the maximum steel strain is given for each analyses. Moreover, shear failure is addressed by means of the required amount of reinforcement. It is noticed that shear failure is strongly affected by the normal force in the different components (beams), see [7].

Depending on the adopted element size, it may be expected that concrete crushing will occur at a compressive concrete strain of about $5 \cdot 10^{-3}$ - $7 \cdot 10^{-3}$, see [7]. Steel failure is expected at a strain of about 5%-7%. Considering these ultimate (failure) strains it may be concluded from Table 14 that failure is expected for all the cases analyzed, except for id02. It may be expected that the damage observed in the Caland tunnel (id02) is rather limited, but for the Leidsche Rijn tunnel failure may be expected. Considering the results of the Caland tunnel and the Drecht tunnel (id01, id02 and id03) it may be expected that the behavior of the Drecht tunnel is comparable to the Caland tunnel. Of course the reinforcement available in the Drecht tunnel is governing the failure behavior.

Table 14 Maximum strains and required amount of shear reinforcement in order to prevent for shear failure (only the non linear analyses are addressed)

criterion	id02	id04	id05	id06
concrete compressive strain	0.65%	1.2%	25%	2%
steel strain (tension/compression)	4.5%	9%	55%	11%
shear failure ($A_{s,req}$ in mm^2/mm) ^{*)}	-	1-1,5	-	-

^{*)} Based on [7]. Id02, id05 and id06 have not been considered with respect to shear failure

Because the current part of the project focuses on a first guess for the expected soil behavior, shear failure of the tunnels is not studied. From [7], however, it may be expected that the nominal shear stress is (momentarily) larger than the shear strength, indicating damage or failure when no reinforcement is present. The amount of required shear reinforcement depends on the geometry of the tunnel. From literature it may be expected that in all tunnels analyzed shear failure/damage may be expected, some times even without the high loading conditions due to an explosion.

It is noticed that the failure assessment of the tunnels is based on several assumptions [7] and relatively simple (beam) models. Moreover, both the soil behavior and 3D effects have not been taken into account and the failure criterion may be debatable.

Finally, as mentioned before, most of the permanent loads on the tunnel are considered to be massless, thereby affecting strongly the dynamic behavior and the shear capacity of the structure. For that reason it is recommended to analyze the failure behavior in more detail. In phase R3 of the project the full system behavior will be taken into account.

4 Summary, conclusions and recommendations

In the Delft Cluster work package “Bijzondere Belastingen” (CT01.21) the consequences of an accident with a transport of explosion hazardous goods are considered: BLEVE and gas explosion. These phenomena have a low probability of occurrence, but might have immense consequences. Therefore, a deterministic consideration is not possible. The current document reports on the first part of research line 2 (structural response) of the project. For studying the soil response due to large deformations (phase 2 of research line 2), finite element simulations have been carried out for studying the effect of high rate loadings on the expected wall and roof deformations. The simulations are based on assumptions that provide for an initial guess with respect to the expected response. In the final phase of the project more detailed analyses are foreseen.

In total three cases, based on the tunnels Caland, Drecht and Leidsche Rijn, the behaviour is studied for three different load scenarios. For all cases a beam model is adopted. Moreover, physical non linear material behavior is taken into account for some of the tunnels analyzed. From the results it was found that for load scenario 1 the amplitude of the walls remains limited (3-7 mm). For load scenario 2 (with a rather high peak load and a high decreasing time/load rate) it was found that failure is to be expected for the Leidsche Rijn tunnel. Also the walls seem to fail. Load scenario 2 was not studied for the Caland and the Drecht tunnel. The roof of the Leidsche Rijn tunnel (having a rather large span) fails for all scenarios considered. Possible shear failure is not analyzed.

The calculated frequency was smaller than 10 Hz for all non linear analyses, in case failure was observed). When no failure was observed the frequency was limited to 30-60 Hz. It is noticed that these conclusions are based on the results obtained with a simple beam model. Because the beam model focuses on the global (bending) behavior, local effects such as shear failure are not taken into account. As a result no high frequency effects are noticed in the analyses. This may be different when using more advanced models. Therefore the results seem valid for structures where bending is dominant and no shear failure is expected. However, when (local) failure of the structure is probable, the results may differ substantially when using more advanced models. These effects will be studied later on in the project (phase 3 of research line 2, see [1]).

5 References

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- [2] C. van der Vliet. Referentieberekening BLEVE-bestendige overkapping. Model-ontwikkeling en rekenresultaten, Rijkswaterstaat, 5 september 2005.
- [3] TNO-rapport 2005-CI-R0094. A2-overkluizing Leidsche Rijn – Beoordeling van de sterkte van de variantoplossingen voor de overkluizing bij explosie ten gevolge van een BLEVE met behulp van DIANA TNO Bouw en Ondergrond; 17 augustus 2005 (in Dutch).
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- [5] A.C. van den Berg. Aanvullende vragen m.b.t. de blastbelasting van de tunnel in de A2 door een BLEVE. TNO rapport 2005-CI-R0172/VTAC, 8 september 2005
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- [7] A.H.J.M. Vervuurt, A.J. Wubs., J.R. van Deursen and J. Weerheijm. Evaluatie van 2 rekenmodellen (SDOF-model en het eindige elementenmodel DIANA) voor de bepaling van de explosiebestendigheid van tunnels. TNO report 2006-D-R0751 (draft). November 28, 2006 (in Dutch).
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- [11] Clough, R.W. en Penzien, J. Dynamics of Structures. 2e druk. McGraw-Hill, Inc. Singapore, 1993.

A DIANA input

A.1 Material properties

```
'MATERIALS'
: concrete properties (in N and m)
  1 YOUNG      31000.0E+06
    POISON     0.15
    DENSIT     2500.0
    TOTCRK     ROTATE
    TENCRCV     LINEAR
    TENSTR     2.33E+06
    GF1        100.0
    CRACKB     0.0625
    COMCRV     CONSTA
    COMSTR     29.8E+06
    SHRCRV     CONSTA

: steel properties (in N and m)
  2 YOUNG      200000.0E+06
    POISON     0.3
    DENSIT     7850.0
    YIELD      VMISES
    YLDVAL     500.0E+06
```

A.2 Command file

```
*FILOS
INITIA
*INPUT
READ          FILE="caland.dat"
READ APPEND   FILE="reinfo.dat"
READ APPEND   FILE="calandp.dat"

*NONLIN
BEGIN TYPE
  PHYSIC OFF
  GEOMET OFF
  BEGIN TRANSI
    BEGIN METHOD
      HHT ALPHA=-0.33
    END METHOD
  DYNAMI
  END TRANSI
END TYPE

: dead weight
BEGIN EXECUT
BEGIN LOAD
  LOADNR=3
  STEPS EXPLIC SIZES 0.05(20)
END LOAD
BEGIN ITERAT
  MAXITE=10
  METHOD NEWTON MODIFI
  BEGIN CONVER
  SIMULT
  FORCE CONTIN
  DISPLA CONTIN
  ENERGY CONTIN
```

```

        END CONVER
        END ITERAT
        END EXECUT

: permanent load (dead weight excluded)
BEGIN EXECUT
  BEGIN LOAD
    LOADNR=1
    STEPS EXPLIC SIZES 0.05(20)
  END LOAD
  BEGIN ITERAT
    MAXITE=10
    METHOD NEWTON MODIFI
  BEGIN CONVER
    SIMULT
    FORCE CONTIN
    DISPLA CONTIN
    ENERGY CONTIN
  END CONVER
  END ITERAT
END EXECUT

: time steps (explosion load)
BEGIN EXECUTE
  BEGIN TIME
    STEPS EXPLIC SIZES 1.0E-05(1000)
  END TIME
  BEGIN ITERAT
    MAXITE=10
    METHOD NEWTON MODIFI
  BEGIN CONVER
    SIMULT
    FORCE CONTIN
    DISPLA CONTIN
    ENERGY CONTIN
  END CONVER
  END ITERAT
END EXECUTE

BEGIN OUTPUT FILE=ID01
  BEGIN SELECT
    STEPS 1-1000000(1)
  END SELECT
  DISPLA TOTAL
  STRESS TOTAL FORCE
  STRESS TOTAL MOMENT
  STRESS TOTAL LOCAL
  STRAIN TOTAL LOCAL
END OUTPUT

BEGIN OUTPUT TABULA FILE="id01-ini.tb"
  BEGIN LAYOUT
    LINPAG 100000
    COLLIN 120
  END LAYOUT
  BEGIN SELECT
    STEPS 1-1000000(1)
    ELEMEN M_ELEM /
    NODES M_NODES /
    BEGIN REINFO ALL /
    ELEMEN M_ELEM /
  END REINFO
  END SELECT
  DISPLA TOTAL X Y
  STRESS TOTAL FORCE X Y
  STRESS TOTAL MOMENT Z

```

```
STRESS TOTAL LOCAL   INTPNT XX
STRAIN TOTAL LOCAL   INTPNT XX
END OUTPUT

BEGIN OUTPUT APPEND FILE=ID01
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    STEPS 50 100 200 250-1000000(250)
  END SELECT
  DISPLA TOTAL
  STRESS TOTAL FORCE
  STRESS TOTAL MOMENT
  STRESS TOTAL LOCAL
  STRAIN TOTAL LOCAL
END OUTPUT

BEGIN OUTPUT TABULA FILE="id01-nlin"
  BEGIN LAYOUT
    LINPAG 100000
    COLLIN 120
  END LAYOUT
  BEGIN SELECT
    STEPS 10-1000000(10)
    ELEMEN  M_ELEM /
    NODES   M_NODES /
    BEGIN REINFO ALL /
      ELEMEN  M_ELEM /
    END REINFO
  END SELECT
  DISPLA TOTAL           X Y
  STRESS TOTAL FORCE     X Y
  STRESS TOTAL MOMENT   Z
  STRESS TOTAL LOCAL    INTPNT XX
  STRAIN TOTAL LOCAL    INTPNT XX
END OUTPUT

*END
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B Results Caland tunnel (id 01)

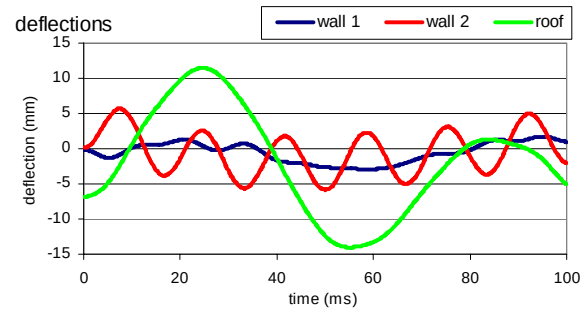


Figure 21 Calculated wall and roof deflections in time (id01: Caland tunnel, linear elastic, load scenario 1)

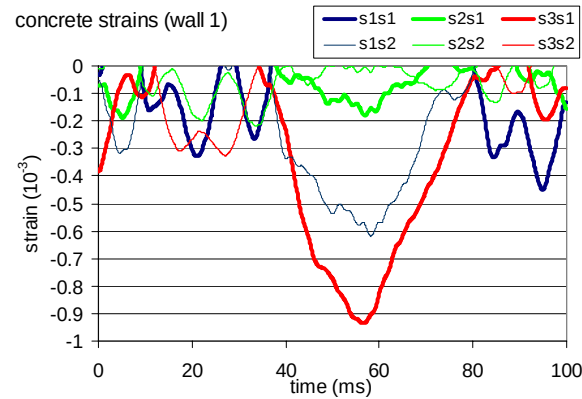


Figure 22 Calculated compressive concrete strains in wall 1 (id01: Caland tunnel, linear elastic, load scenario 1)

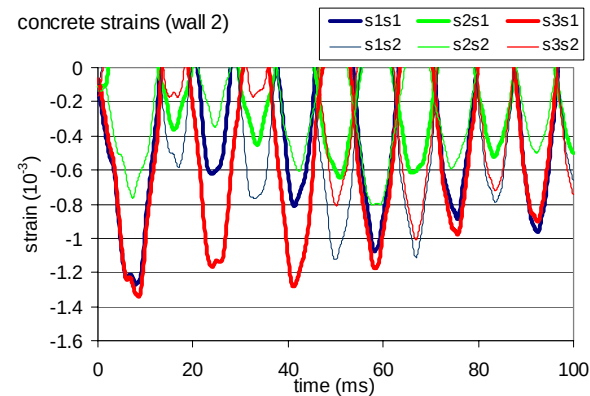


Figure 23 Calculated compressive concrete strains in wall 2 (id01: Caland tunnel, linear elastic, load scenario 1)

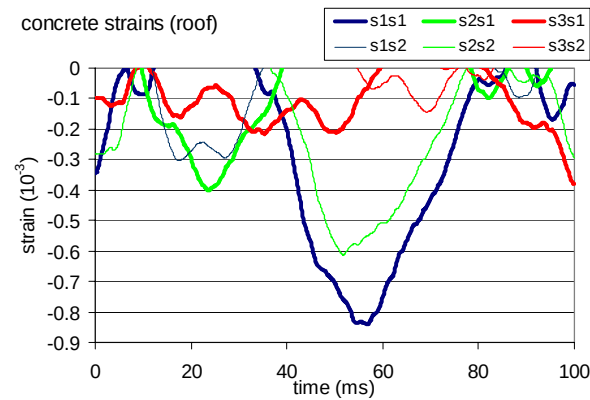


Figure 24 Calculated compressive concrete strains in the roof (id01: Caland tunnel, linear elastic, load scenario 1)

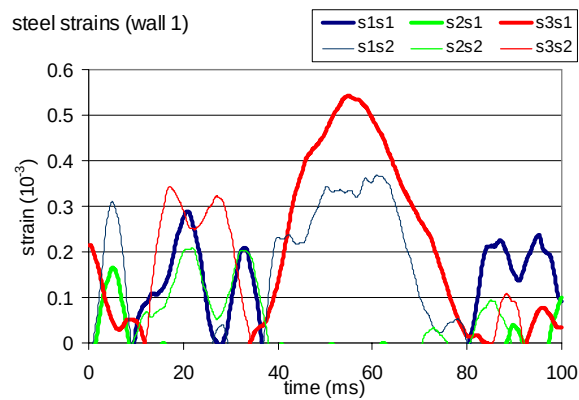


Figure 25 Calculated steel strains in wall 1 (id01: Caland tunnel, linear elastic, load scenario 1)

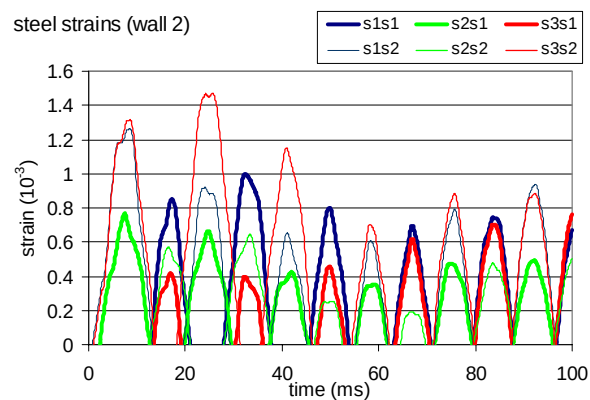


Figure 26 Calculated steel strains in wall 2 (id01: Caland tunnel, linear elastic, load scenario 1)

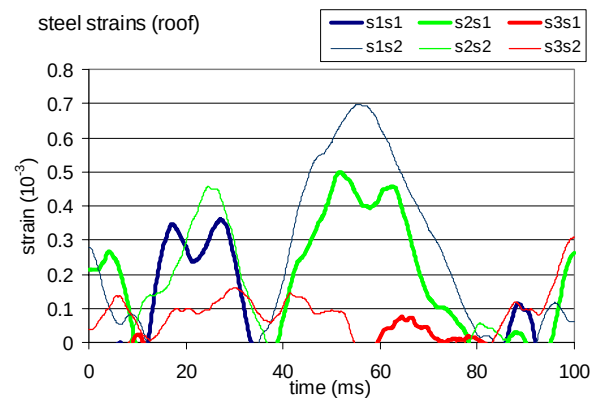


Figure 27 Calculated steel strains in the roof (id01: Caland tunnel, linear elastic, load scenario 1)

C Results Caland tunnel (id 02)

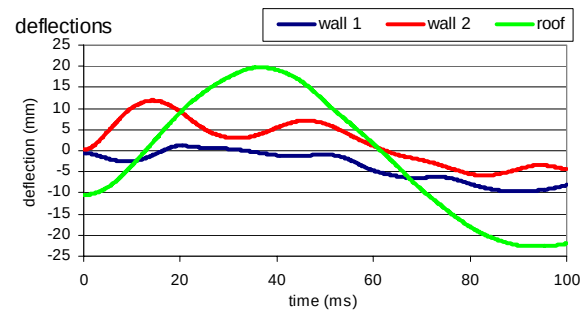


Figure 28 Calculated wall and roof deflections in time (id02: Caland tunnel, non linear, load scenario 1)

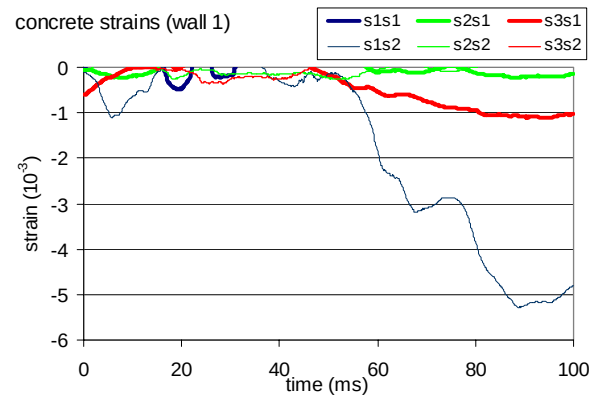


Figure 29 Calculated compressive concrete strains in wall 1 (id02: Caland tunnel, non linear, load scenario 1)

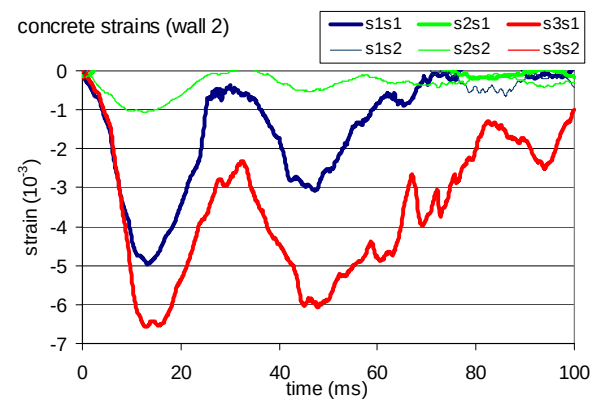


Figure 30 Calculated compressive concrete strains in wall 2 (id02: Caland tunnel, non linear, load scenario 1)

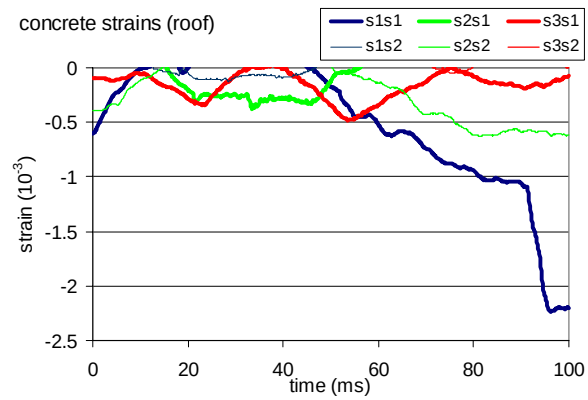


Figure 31 Calculated compressive concrete strains in the roof (id02: Caland tunnel, non linear, load scenario 1)

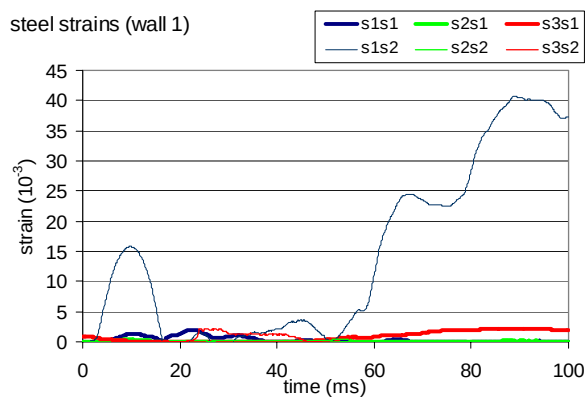


Figure 32 Calculated steel strains in wall 1 (id02: Caland tunnel, non linear, load scenario 1)

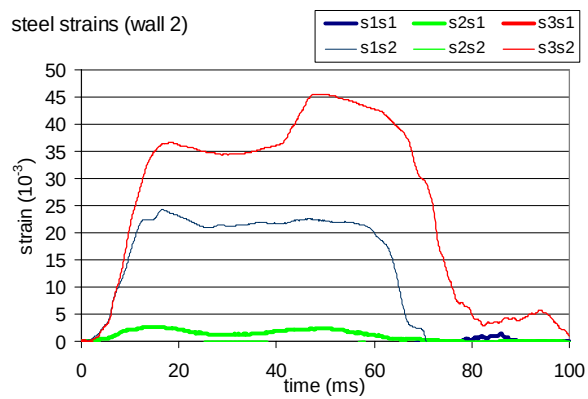


Figure 33 Calculated steel strains in wall 2 (id02: Caland tunnel, non linear, load scenario 1)

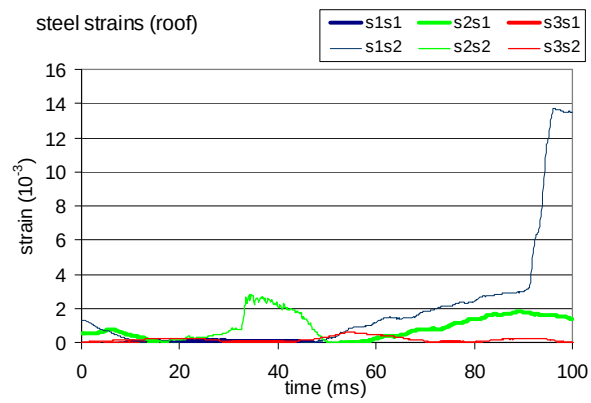


Figure 34 Calculated steel strains in the roof (id02: Caland tunnel, non linear, load scenario 1)

D Results Drecht tunnel (id 03)

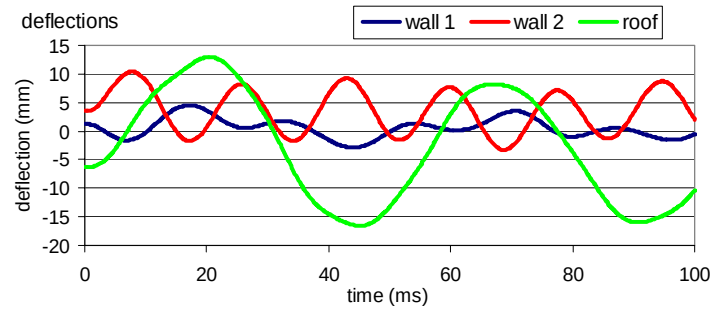


Figure 35 Calculated wall and roof deflections in time (id03: Drecht tunnel, linear elastic, load scenario 1)

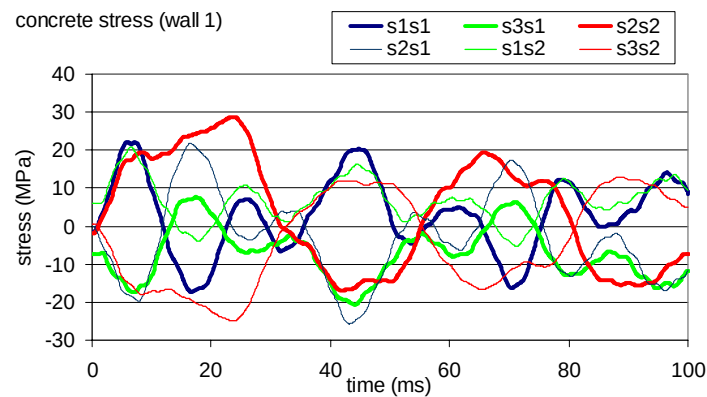


Figure 36 Calculated concrete stress in wall 1 (id03: Drecht tunnel, linear elastic, load scenario 1)

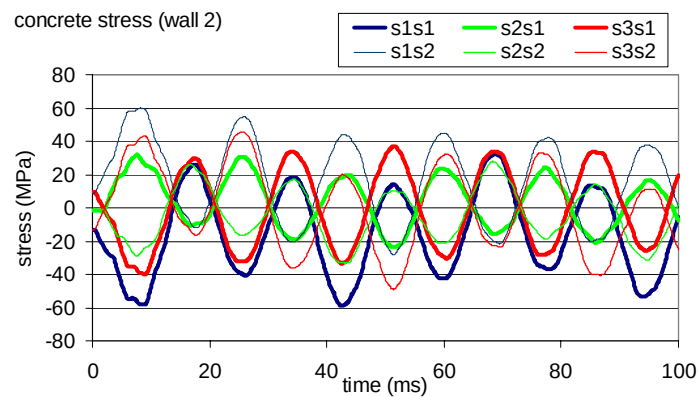


Figure 37 Calculated concrete stress in wall 2 (id03: Drecht tunnel, linear elastic, load scenario 1)

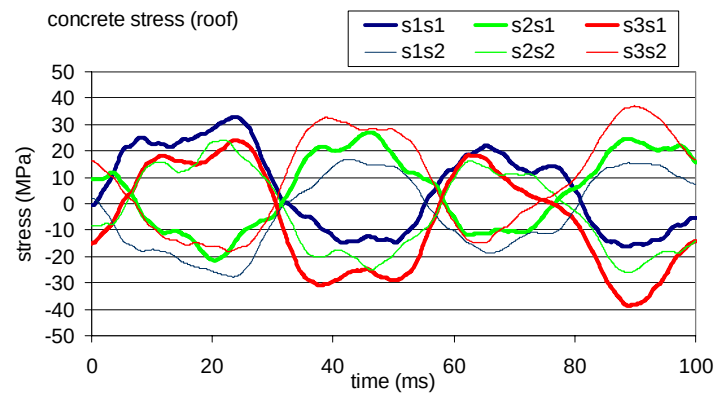


Figure 38 Calculated concrete stress in the roof (id03: Drecht tunnel, linear elastic, load scenario 1)

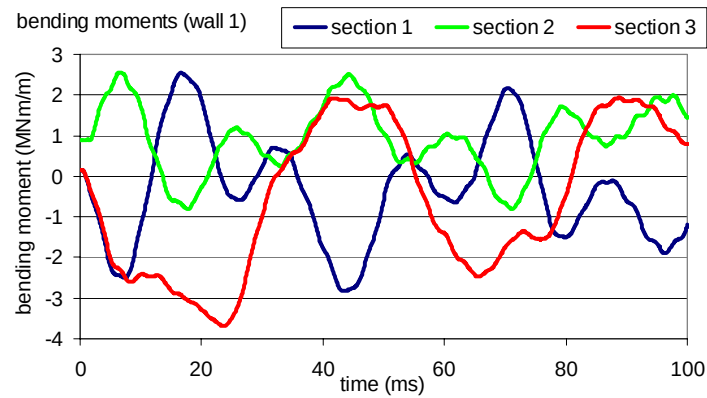


Figure 39 Calculated bending moments in wall 1 (id03: Drecht tunnel, linear elastic, load scenario 1)

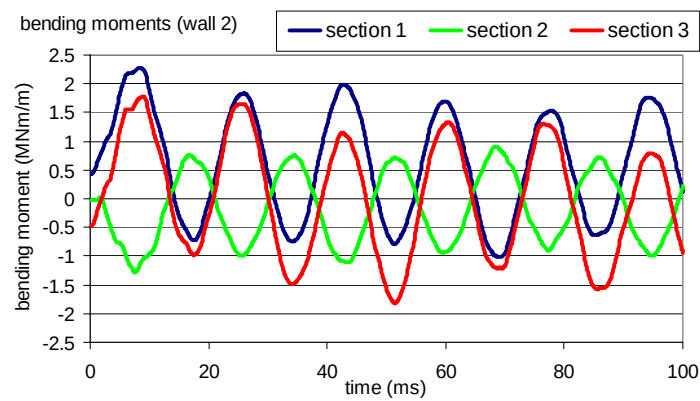


Figure 40 Calculated bending moments in wall 2 (id03: Drecht tunnel, linear elastic, load scenario 1)

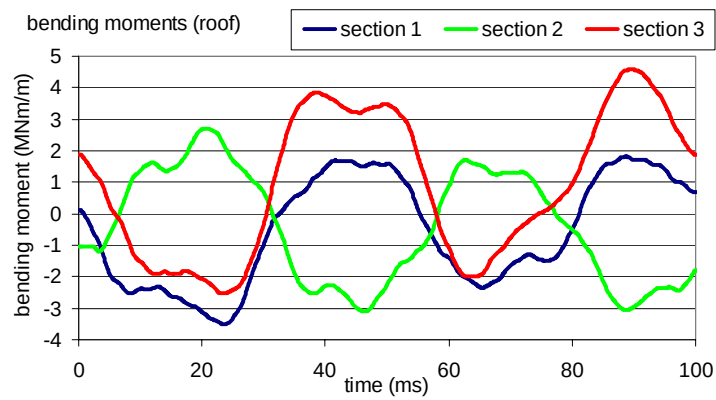


Figure 41 Calculated bending moments in the roof (id03: Drecht tunnel, linear elastic, load scenario 1)

E Results Leidsche Rijn tunnel (id 04)

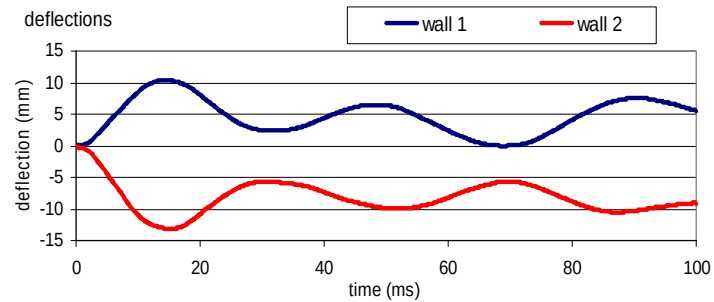


Figure 42 Calculated wall deflections in time (id04: Leidsche Rijn tunnel, non linear, load scenario 1)

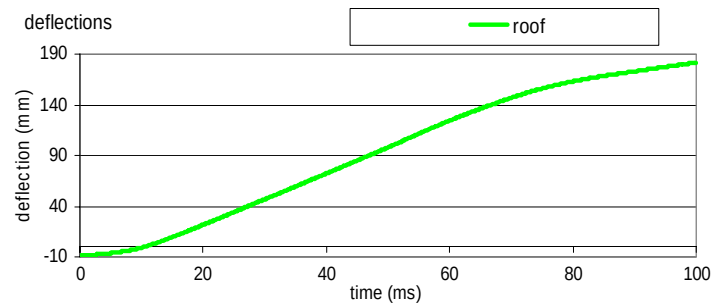


Figure 43 Calculated roof deflections in time (id04: Leidsche Rijn tunnel, non linear, load scenario 1)

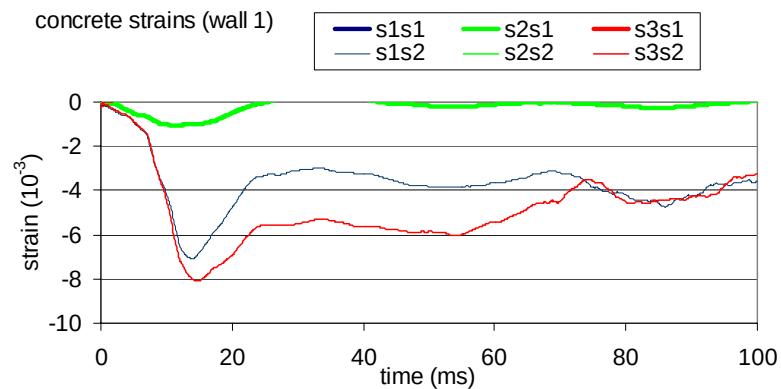


Figure 44 Calculated compressive concrete strains in wall 1 (id04: Leidsche Rijn tunnel, non linear, load scenario 1)

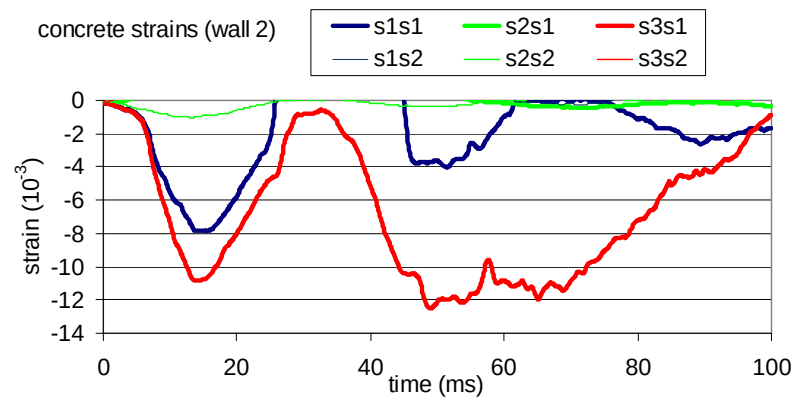


Figure 45 Calculated compressive concrete strains in wall 2 (id04: Leische Rijn tunnel, non linear, load scenario 1)

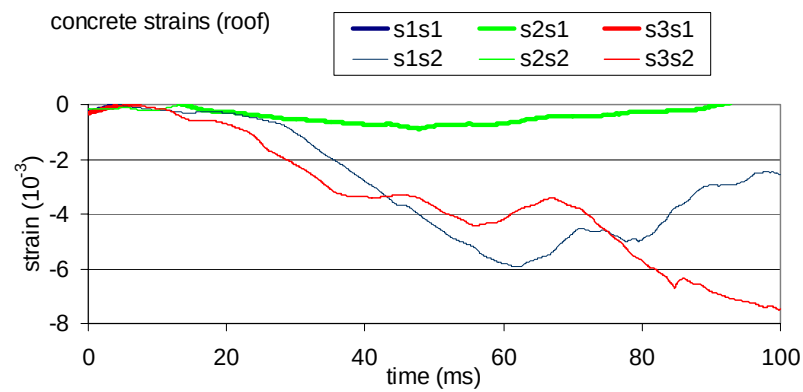


Figure 46 Calculated compressive concrete strains in the roof (id04: Leische Rijn tunnel, non linear, load scenario 1)

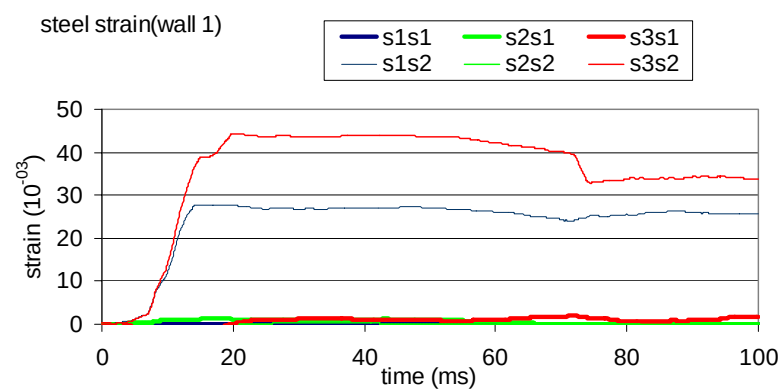


Figure 47 Calculated steel strains in wall 1 (id04: Leische Rijn tunnel, non linear, load scenario 1)

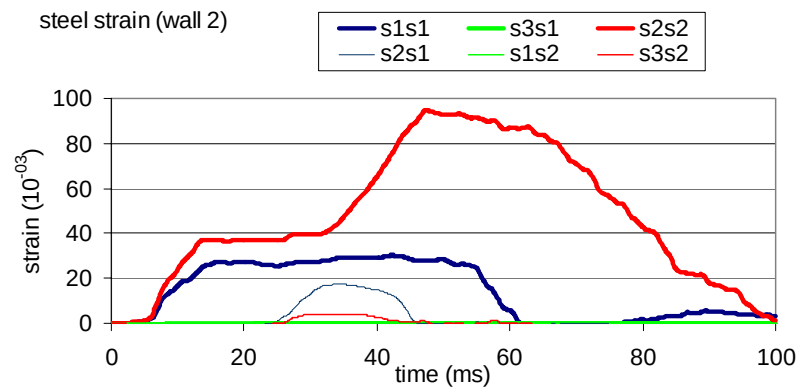


Figure 48 Calculated steel strains in wall 2 (id04: Leische Rijn tunnel, non linear, load scenario 1)

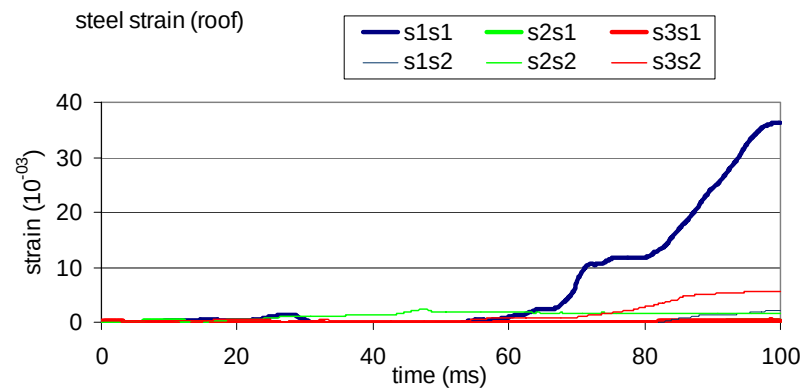


Figure 49 Calculated steel strains in the roof (id04: Leische Rijn tunnel, non linear, load scenario 1)

F Results Leidsche Rijn tunnel (id 05)

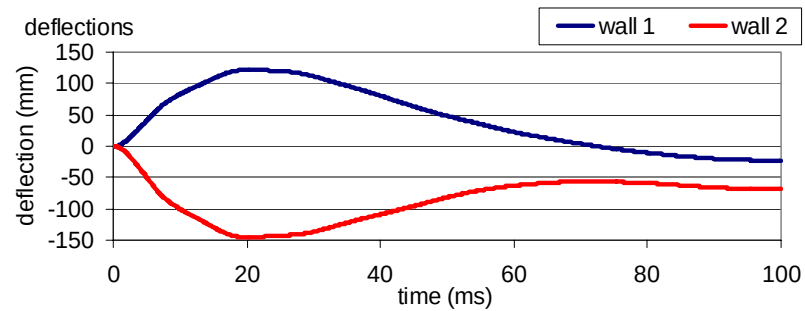


Figure 50 Calculated wall deflections in time (id05: Leidsche Rijn tunnel, non linear, load scenario 2)

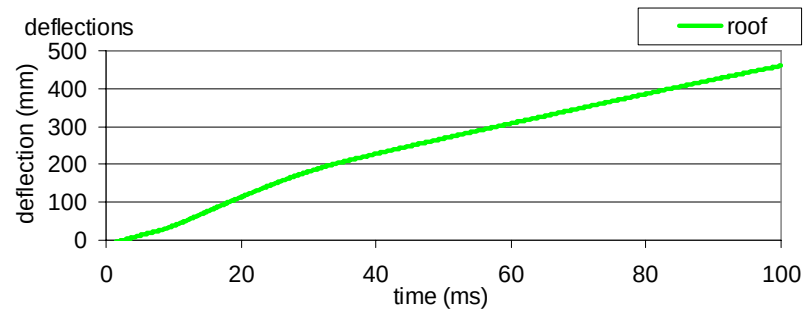


Figure 51 Calculated roof deflections in time (id05: Leidsche Rijn tunnel, non linear, load scenario 2)

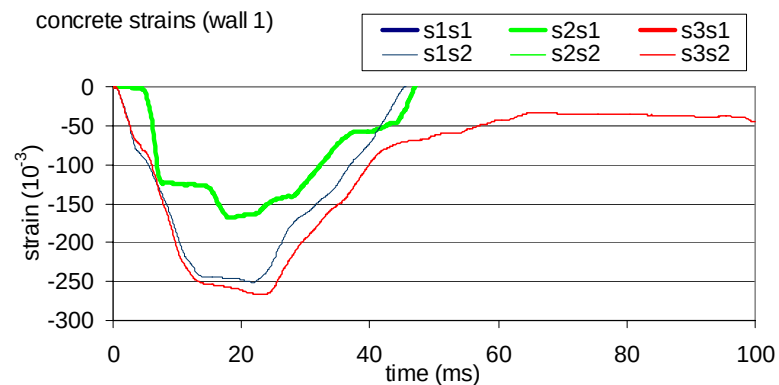


Figure 52 Calculated compressive concrete strains in wall 1 (id05: Leidsche Rijn tunnel, non linear, load scenario 2)

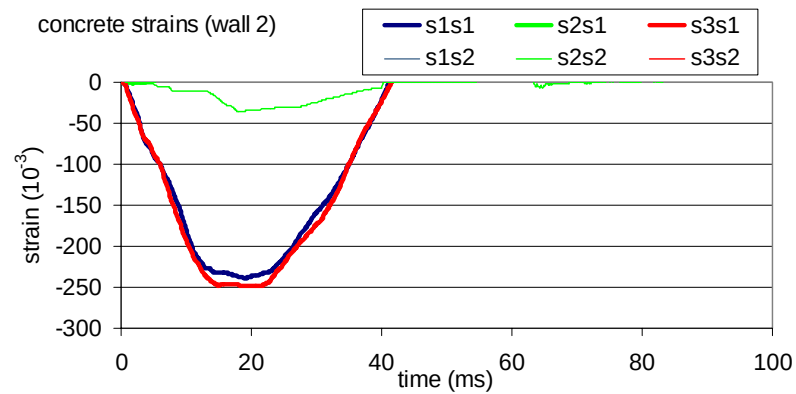


Figure 53 Calculated compressive concrete strains in wall 2 (id05: Leische Rijn tunnel, non linear, load scenario 2)

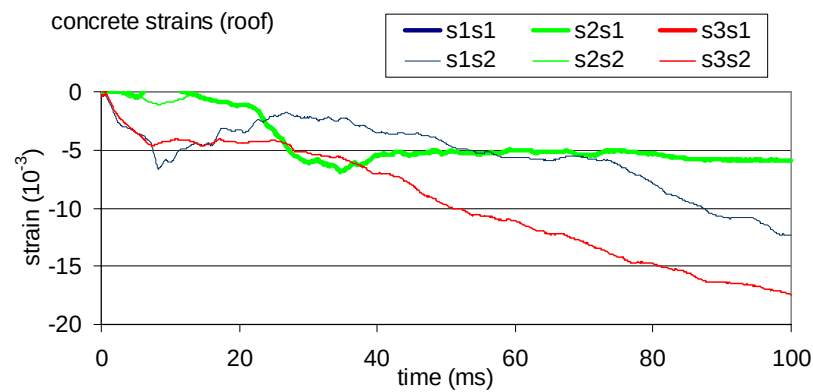


Figure 54 Calculated compressive concrete strains in the roof (id05: Leische Rijn tunnel, non linear, load scenario 2)

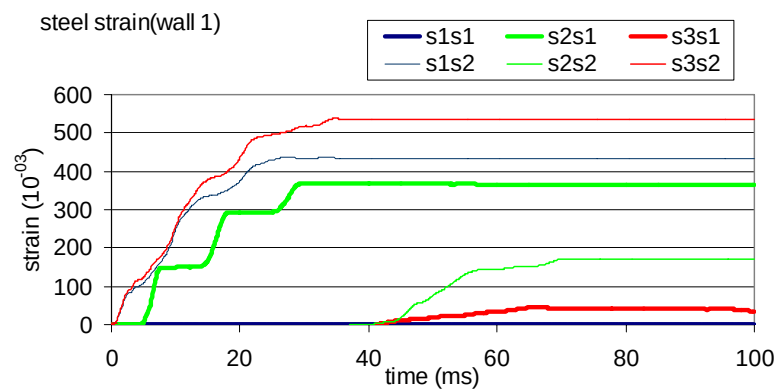


Figure 55 Calculated steel strains in wall 1 (id05: Leische Rijn tunnel, non linear, load scenario 2)

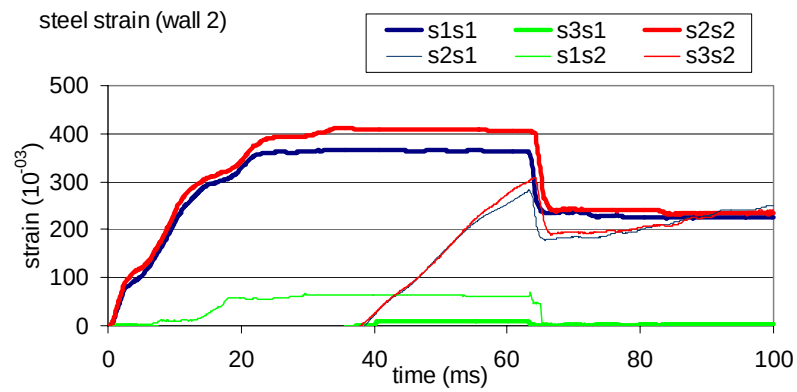


Figure 56 Calculated steel strains in wall 2 (id05: Leische Rijn tunnel, non linear, load scenario 2)

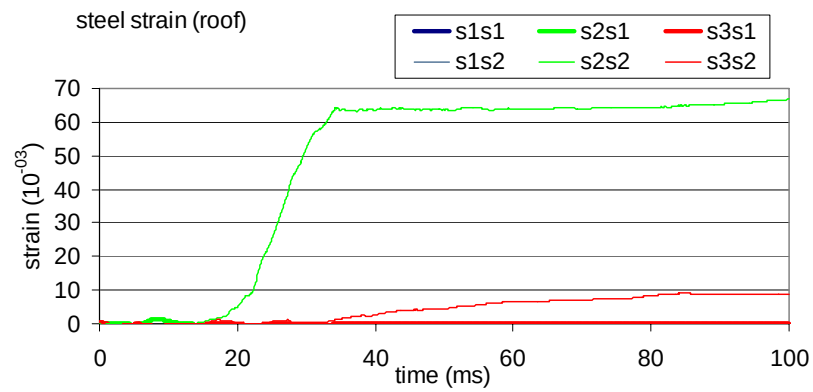


Figure 57 Calculated steel strains in the roof (id05: Leische Rijn tunnel, non linear, load scenario 2)

G Results Tunnel Leidsche Rijn tunnel (id 06)

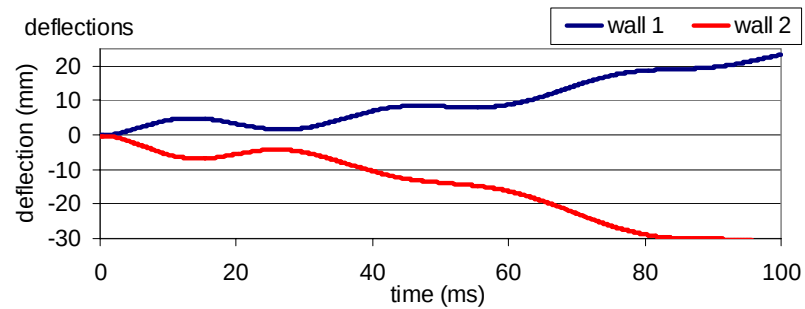


Figure 58 Calculated wall deflections in time (id06: Leidsche Rijn tunnel, non linear, load scenario 3)

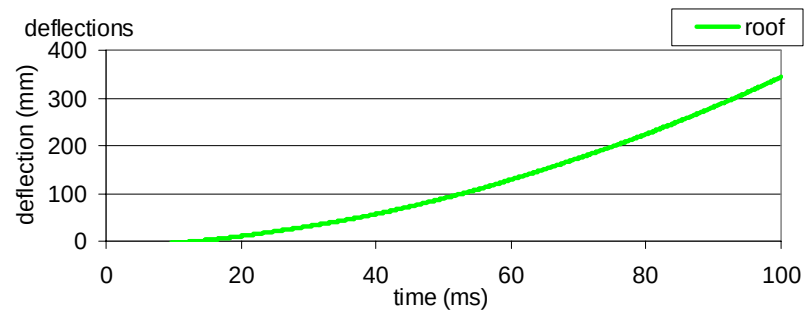


Figure 59 Calculated roof deflections in time (id06: Leidsche Rijn tunnel, non linear, load scenario 3)

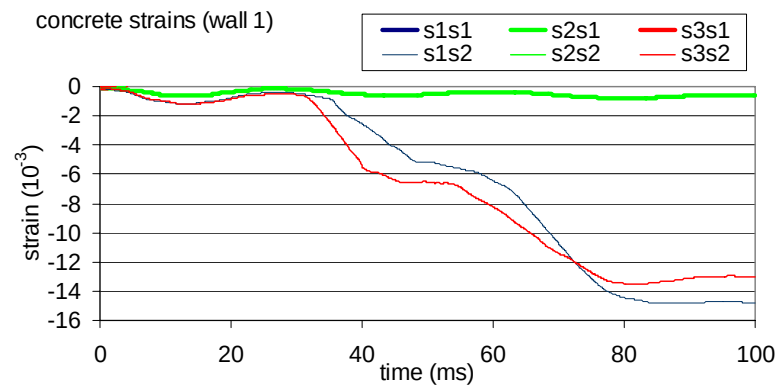


Figure 60 Calculated compressive concrete strains in wall 1 (id06: Leidsche Rijn tunnel, non linear, load scenario 3)

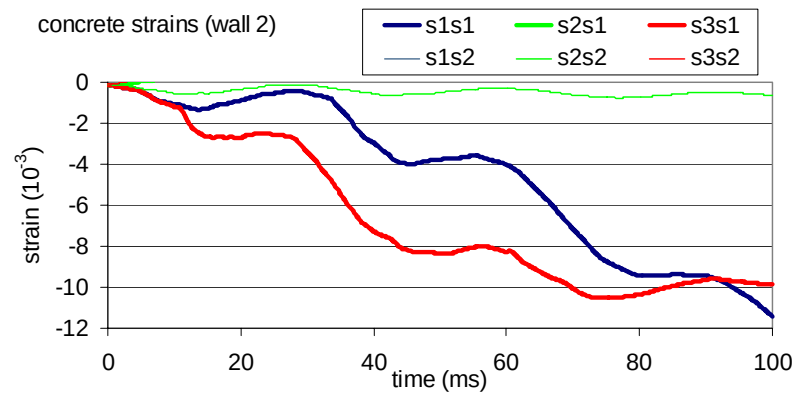


Figure 61 Calculated compressive concrete strains in wall 2 (id06: Leische Rijn tunnel, non linear, load scenario 3)

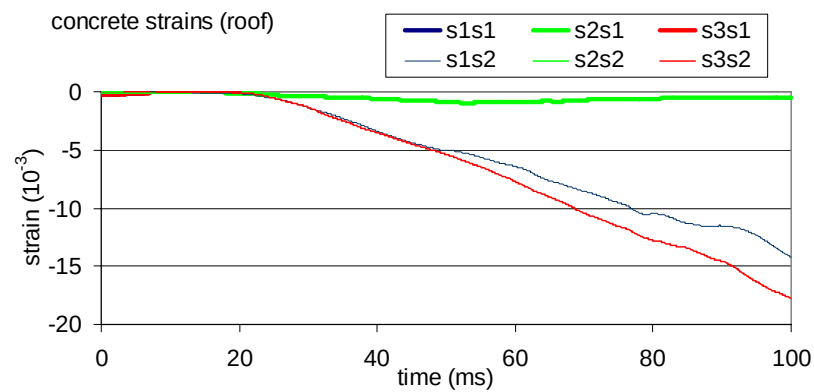


Figure 62 Calculated compressive concrete strains in the roof (id06: Leische Rijn tunnel, non linear, load scenario 3)

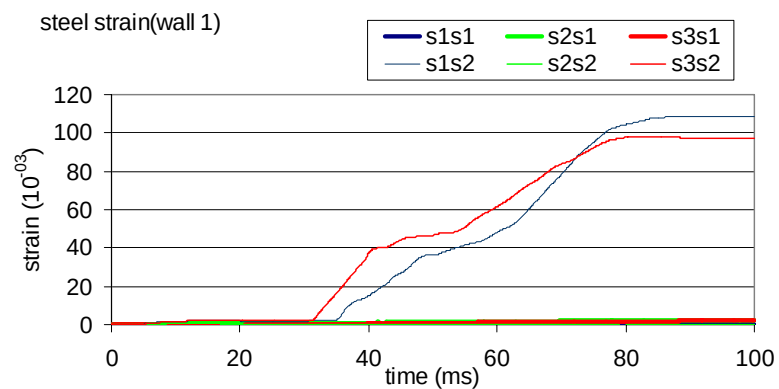


Figure 63 Calculated steel strains in wall 1 (id06: Leische Rijn tunnel, non linear, load scenario 3)

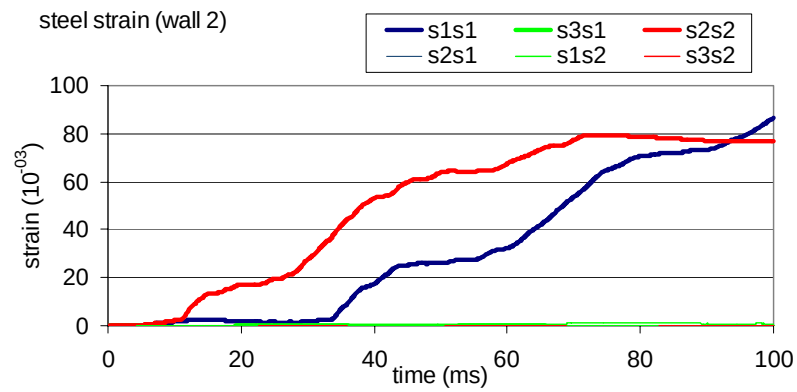


Figure 64 Calculated steel strains in wall 2 (id06: Leische Rijn tunnel, non linear, load scenario 3)

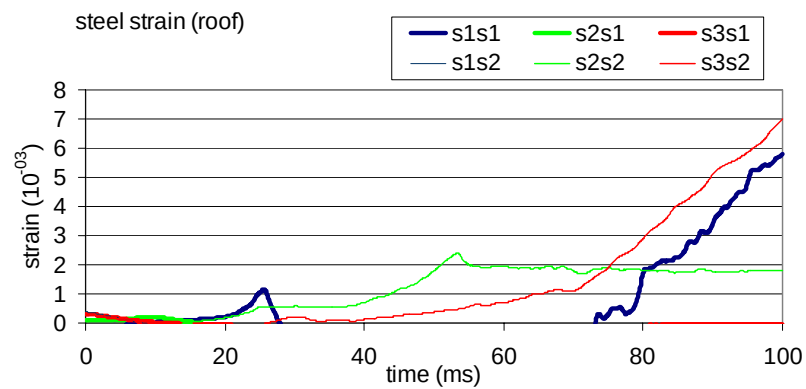


Figure 65 Calculated steel strains in the roof (id06: Leische Rijn tunnel, non linear, load scenario 3)

H Simulation of the tunnel response using a 2D membrane model, including the effect of the soil

H.1 Introduction

The analyses which have been documented in the report have been carried out under the assumption that a beam model may be used for analysing the structure and that the soil surrounding the tunnel only causes an external loading on the tunnel (in the form of soil pressure). The effect of soil mass and stiffness has been neglected and the bottom of the tunnel was assumed to be rigidly supported. Presumably such assumptions, lead to an overestimation of forces in the tunnel structure. Further, these simulations provide no insight to the stresses which occur in the soil. In order to obtain insight into the response of the tunnel and the soil, analyses have been carried out with a 2D membrane model in which the surrounding soil is modelled. Moreover, in Annex H.5, the effect of taking into account the soil in the membrane model is studied briefly. A comparison of the response in this analysis and those described in the report will show what differences are caused when the soil mass and stiffness may be neglected.

H.2 Model description

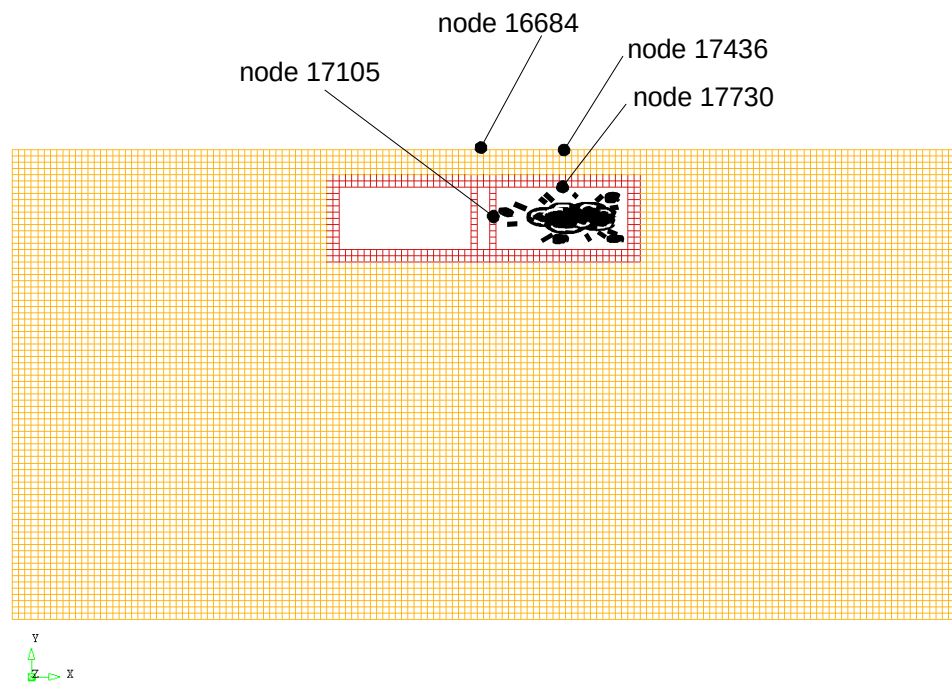


Figure 66 Finite element model

The model of the tunnel is based on the geometry of the Caland Tunnel. The problem is completely fictitious, i.e. no attempt to model the tunnel and the soil as in the real situation has been made. The size of tunnel elements such as walls and floor is a multiple of a standard element size of 0.6 by 0.6 m. Quadrilateral 8 node plane strain elements have been used to make the mesh. The tunnel main walls and floors have a thickness of 1.2 m. The inside walls have a size of 0.6 m. It is noted that this is different from the beam model ($t=0,5$ m). The tunnel is situated 2.4 m under the soil surface. The

complete model has a cross section of 90 m by 48 m. Silent boundary elements have been applied around the left, bottom and right boundaries. These elements absorb only incoming dilatational waves. The upper edge of the model is free.

The weight of the model is not taken into account. Because the material behaviour is linear elastically, superposition of the static weight response may be applied. The mass, on the other hand, is taken into account for all model components. For the soil a mass, corresponding to wet conditions are assumed. The mass and the weight of the water on top of the soil is not taken into account. The effects of these assumptions may differ in time and are not studied in this report.

For comparison to the beam model, an additional analysis with the membrane model is carried out in which the soil is neglected and the bottom of the tunnel is fully supported (corresponding to the assumption in the analyses with the beam model). The results of this additional analysis are presented in Annex H.5. The remainder of this annex focuses on the membrane model including soil.

Material properties

The material properties adopted in the analysis are given in Table 15.

Table 15 Material properties adopted in the analysis

Material number	Material	Young's modulus	Poisson ratio	Density	Damping ratio
		[MPa]	[-]	[kg/m ³]	ζ [-]
1	Sand	100	0.3	2000	2.5%
2	Concrete	31000	0.15	2500	1.5%

Damping

In dynamic FEM calculations use is made of the following equation of motion:

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{F} \quad (1)$$

where:

- $\mathbf{M}$ = mass matrix
- $\mathbf{C}$ = damping matrix
- $\mathbf{K}$ = stiffness matrix
- $\mathbf{u}$ = displacement vector
- $\mathbf{F}$ = external force vector

The damping matrix is then filled with values of the viscous damping, which rules a force counteracting the velocity. The damping matrix $\mathbf{C}$ is not easily incorporated into the finite element solution unless simplifying assumptions are made. Subsequently the concept of Rayleigh damping is introduced. It makes use of two damping parameters, η_1 and η_2 , respectively coupled to the mass matrix and the stiffness matrix:

$$\mathbf{C} = \eta_1 \mathbf{M} + \eta_2 \mathbf{K} \quad (2)$$

This type of damping gives rise to a system with proportional damping, whereby, the coordinate transformation that diagonalizes the mass and stiffness matrices also diagonalizes the damping matrix. The associated relationship between the damping ratio and frequency is given by:

$$D(\omega) = \frac{\eta_1}{2\omega} + \frac{\eta_2\omega}{2} \quad (3)$$

In Figure 67 the variation of the damping ratio with frequency is shown.

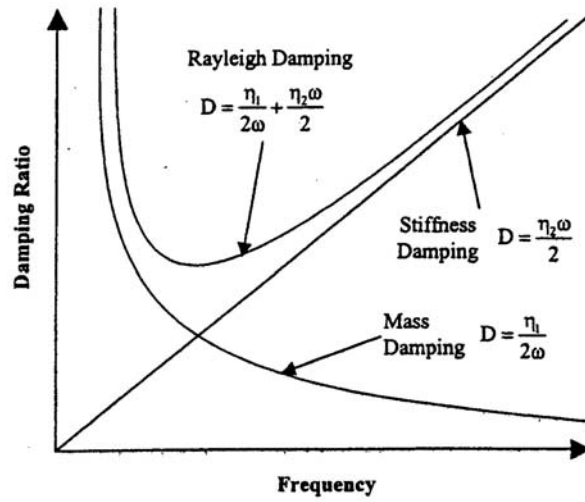


Figure 67: Relationship between Rayleigh damping parameters and damping ratio.

If the damping ratio for two different (control) frequencies is known, then by using Eq. 3, corresponding values of η_1 and η_2 can be calculated. Often, detailed information about the variation of damping ratio with frequency is not available, hence it is usually assumed that the same damping ratio applies to both control frequencies, ω_m and ω_n . In this case the damping factors η_1 and η_2 can be determined using the following expression, Clough and Penzien, [11]:

$$\begin{Bmatrix} \eta_1 \\ \eta_2 \end{Bmatrix} = \frac{2D}{\omega_m + \omega_n} \begin{Bmatrix} \omega_m\omega_n \\ 1 \end{Bmatrix} \quad (4)$$

where D is the damping ratio at the selected control frequencies.

In applying Rayleigh damping, care must be taken so as to choose damping factors such that the resulting damping ratio between the frequencies of interest is similar to that required in the model. It should be kept in mind that the damping ratio will be lower than that supplied in Eq. 4 between the two control frequencies and that outside this range the damping ratio will be higher.

The applied Rayleigh damping parameters are given in Table 16. The parameters are determined such that the resulting damping at 0.5 Hz and 10 Hz is equal to the selected

damping ratio of the material (as given in the table). The damping ratios are based on engineering judgement.

The theoretical form of the load is defined as $y=y_0 \cdot \exp(-2\pi f_{bo} t)$ where f_{bo} is can be considered as a break off frequency, that is, a frequency above which the signal strength decreases. For the given loading, the break off frequency is about 10 Hz. This is elucidated in Figure 68. The red line shows the theoretical form for the applied load that is used for determining the Raleigh damping ratios. The bleu line indicates the load that is applied in the analyses.

Table 16 Rayleigh damping parameters

Material number	Material	Damping ratio	Frequency range [Hz]	
		ζ [-]	η_1	η_2
1	Sand	2.5%	1.5E-01	7.6E-04
2	Concrete	1.5%	9.0E-02	4.5E-04

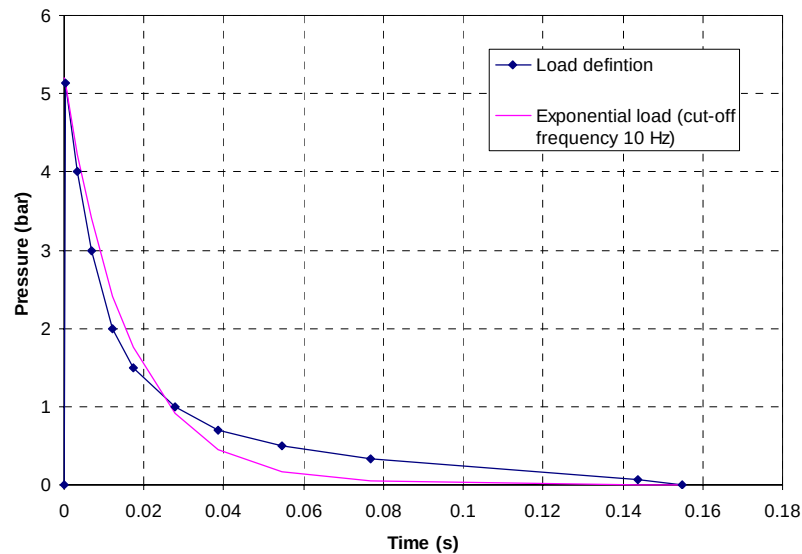


Figure 68 Loading history.

Loading

The same explosion load is used as in the original analysis in the report, see section 2.3.3. This loading has a nearly exponential form as can be seen from the diagram in Figure 68. The actually applied load in the analyses corresponds to the bleu dotted line in Figure 68. The load is applied as a uniform pressure along the inside perimeter of the right tunnel section.

Element size

In wave propagation problems, element dimensions are chosen with respect to the highest frequency of the wave with the lowest velocity. Element dimensions that are too large will filter high frequencies, whereas very small element dimensions can introduce numerical instability as well as require considerable computational resources. An approximate element dimension ($D_{\max}$) is calculated using

$$D_{\max} = X\lambda_{\min} \quad (5)$$

where $\lambda_{\min}$ is the minimum wavelength. The constant, X , must be less than 0.5 because of the Nyquist limit, and further it depends on whether the mass matrices are consistent ($X=0.25$) or lumped ($X=0.2$). This formulation assumes that elements have square dimensions (Valliappan and Murti, Saenger et al.). The minimum wavelength is determined by the slowest waves which need to be modeled and the highest frequency of interest:

$$\lambda_{\min} = \frac{c_{\min}}{f_{\max}} \quad (6)$$

Different wave types can occur in solids. For the problem being considered here, the most relevant are the longitudinal, the transverse and the Rayleigh wave types, the latter type being the slowest. Wave speeds depend on the Young's modulus, the Poisson ratio and the density of the material. Formulas for calculating wave speeds can be found in standard textbooks, for example, Cremer et al., [9].

A single element size has been selected of 0.6 m for all elements. In Table 17 the maximum element sizes are calculated. Generally it is assumed that five elements in a row are sufficient to model a complete wave. At 10 Hz, wavelengths of 13 m appear in the soil. An element size of 0.6 m would be in fact sufficient to model waves with a frequency up to more than 40 Hz.

Table 17 Determination of the maximum element size

Material	Symbol	Density Unit [kg/m ³]	Young's modulus [Pa]	Poisson ratio [-]	Maximum frequency f_max [Hz]	Longitudinal wave speed C _{LII} [m/s]	Transverse (shear) wave speed C _T [m/s]	Rayleigh wave speed C _R [m/s]	0.2*λ_min D_max [m]
Sand		2000	1.00E+08	0.33	10	272	137	128	2.553
Concrete		2500	3.10E+10	0.15	10	3618	2322	2086	41.714

H.3 Analysis

A transient analysis with Newmark time integration (with $\beta=0.25$ and $\gamma=0.5$) has been carried out. The time step, Δt , is 1 ms and the total simulation time is 0.5 s (500 steps).

H.4 Results

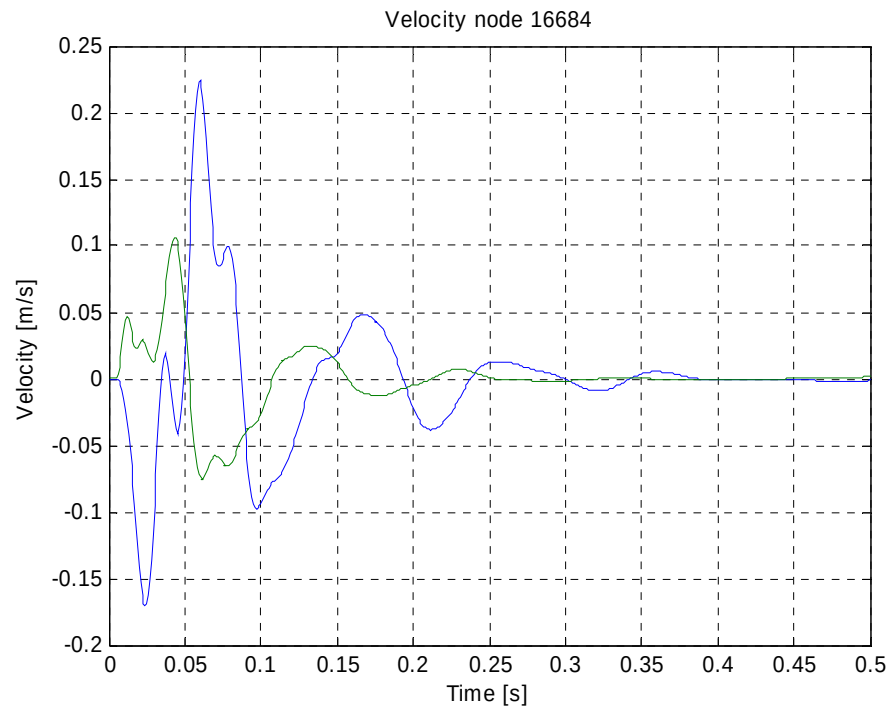


Figure 69 Velocity time history of central node at surface level (top edge of model). Blue line: horizontal velocity; green line: vertical velocity.

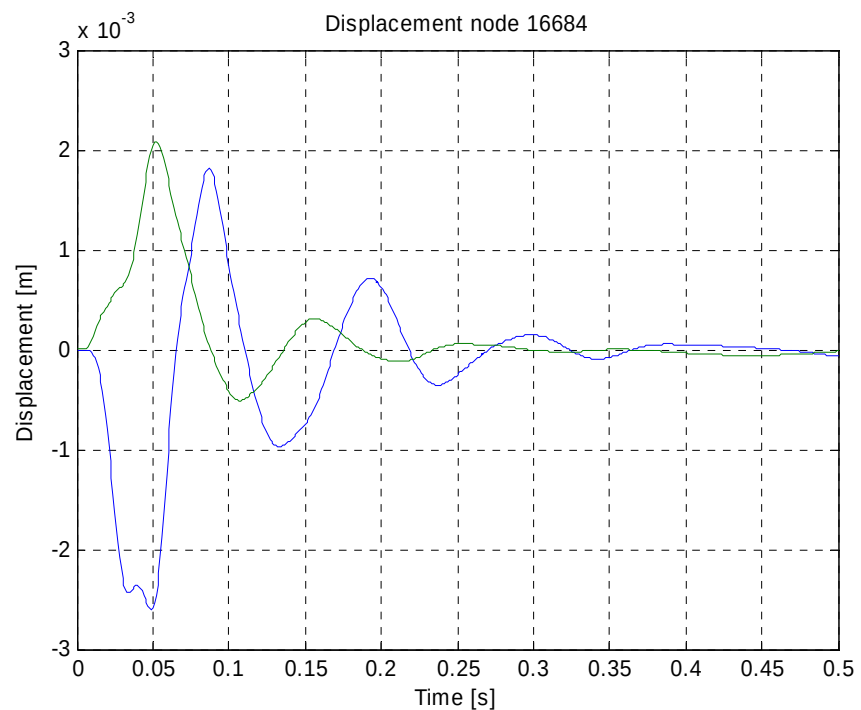


Figure 70 Displacement time history of central node at surface level (top edge of model). Blue line: horizontal displacement; green line: vertical displacement.

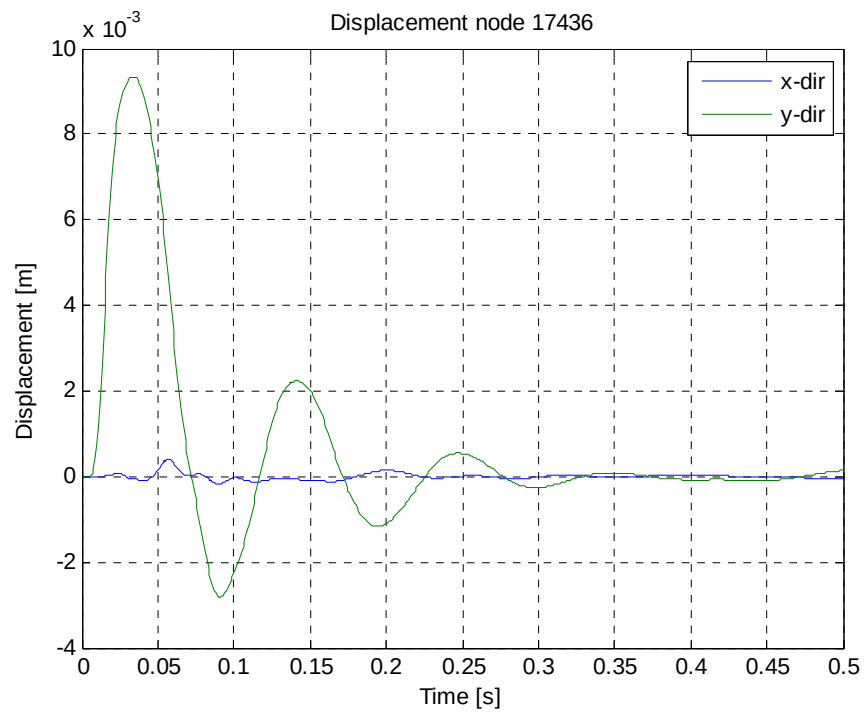


Figure 71 Displacement time history of surface directly above right tunnel section.
Blue line: horizontal displacement; green line: vertical displacement.

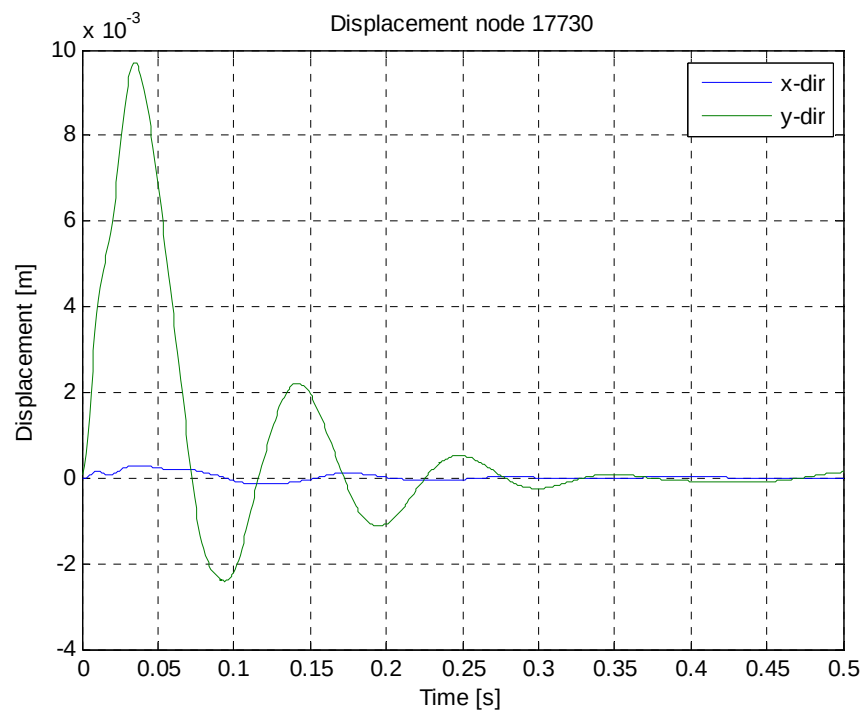


Figure 72 Displacement time history of central node in ceiling of loaded tunnel section.

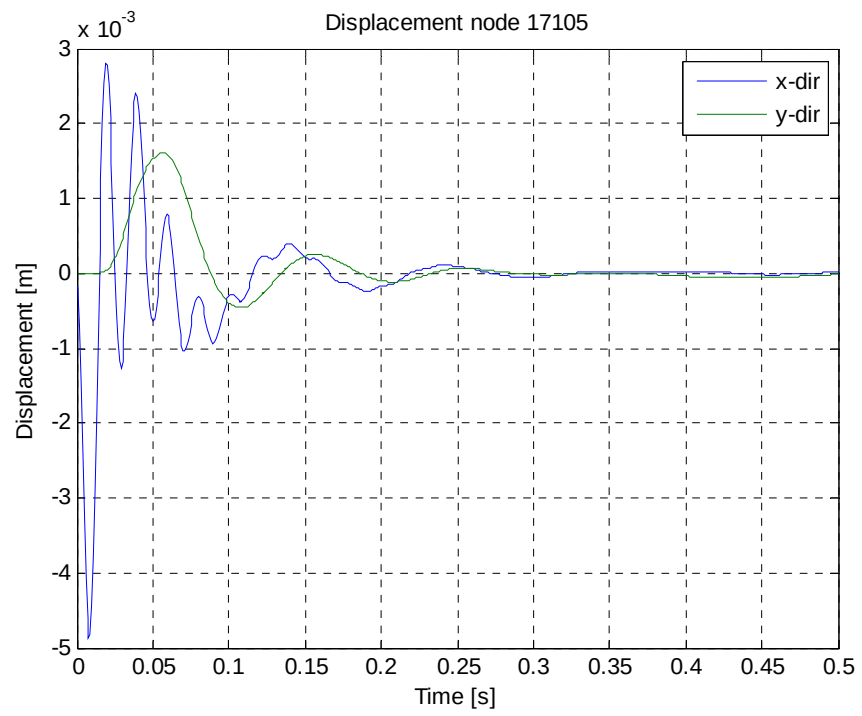


Figure 73 Displacement time history in the middle of the right separation wall.

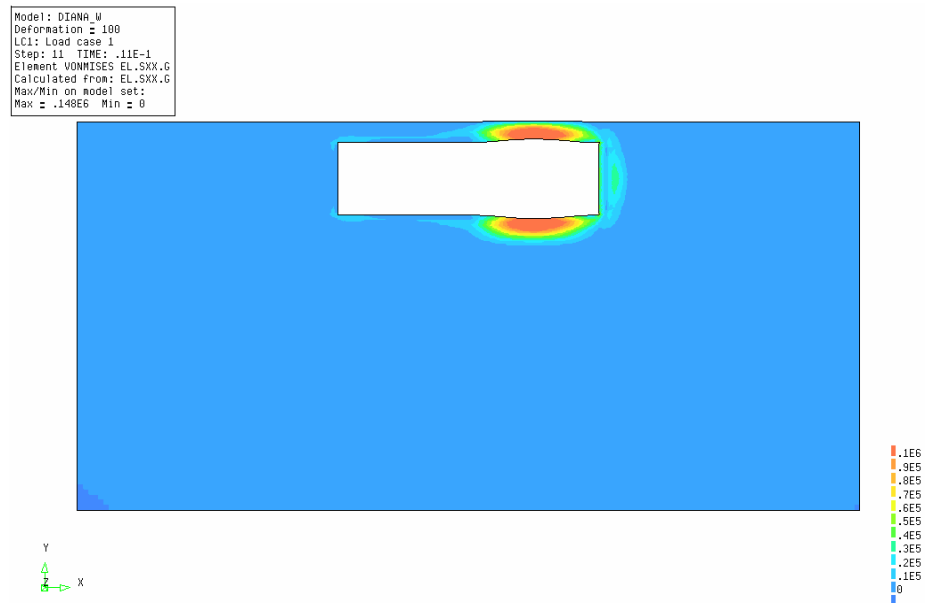


Figure 74 Von Mises stresses in soil at 11 ms

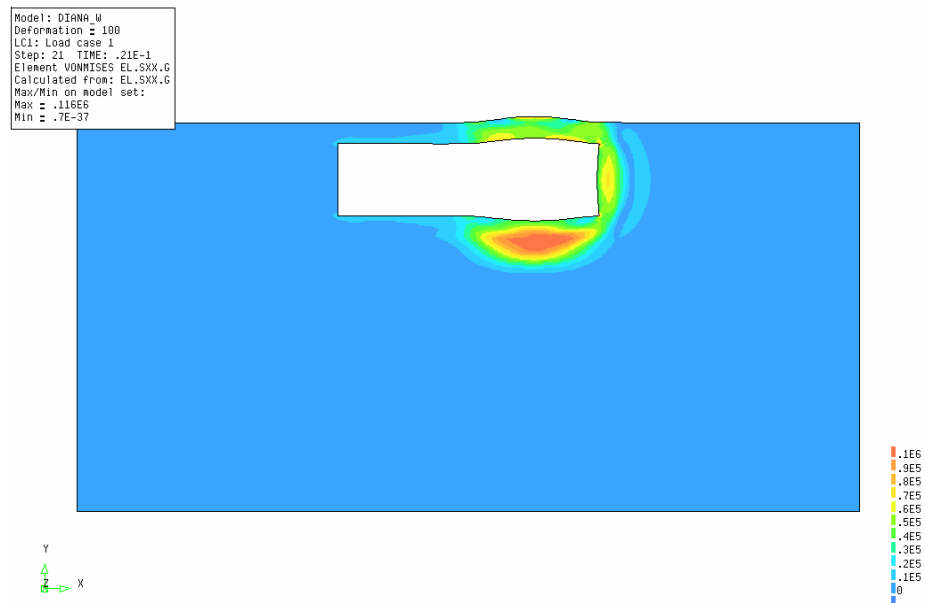


Figure 75 Von Mises stresses in soil at 21 ms

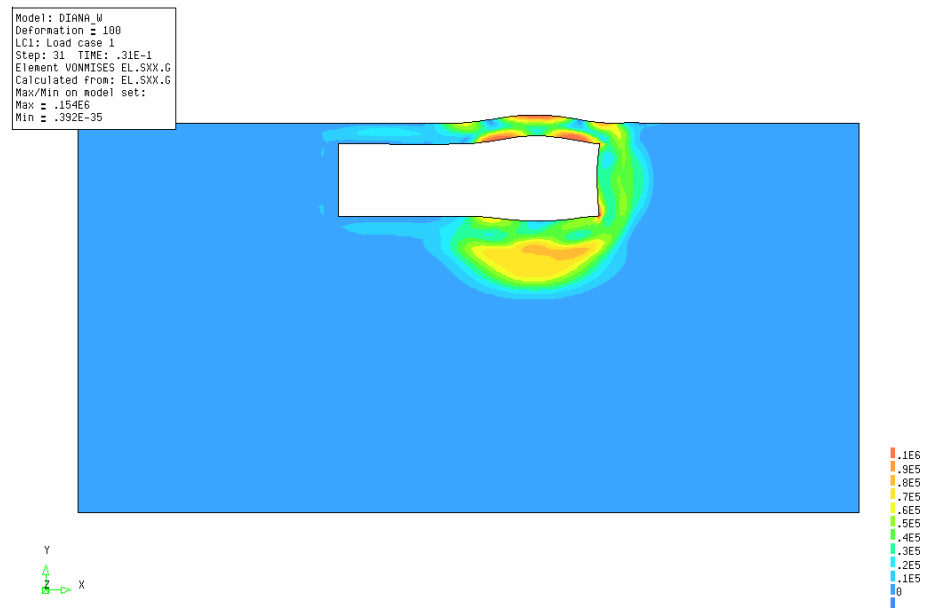


Figure 76 Von Mises stresses in soil at 31 ms

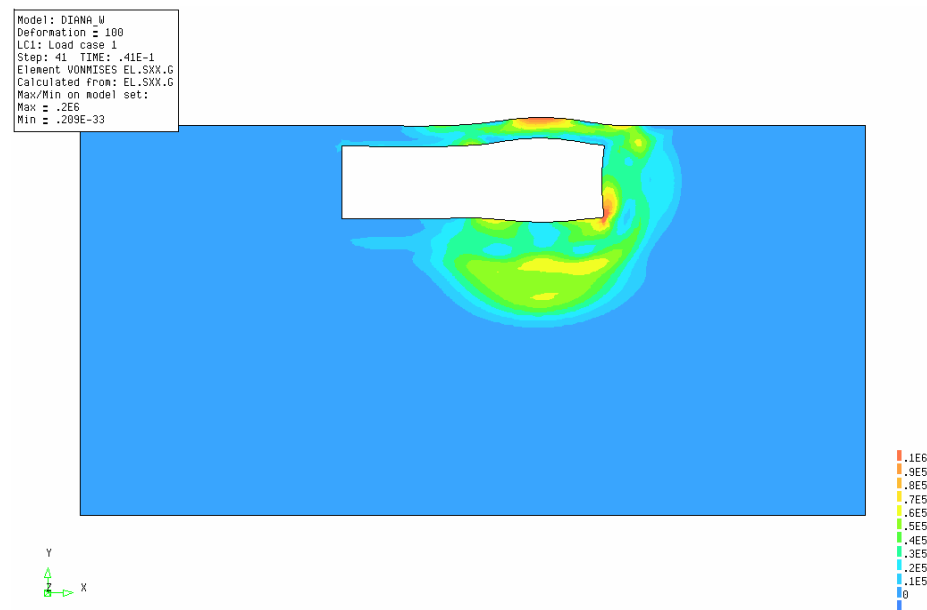


Figure 77 Von Mises stresses in soil at 41 ms

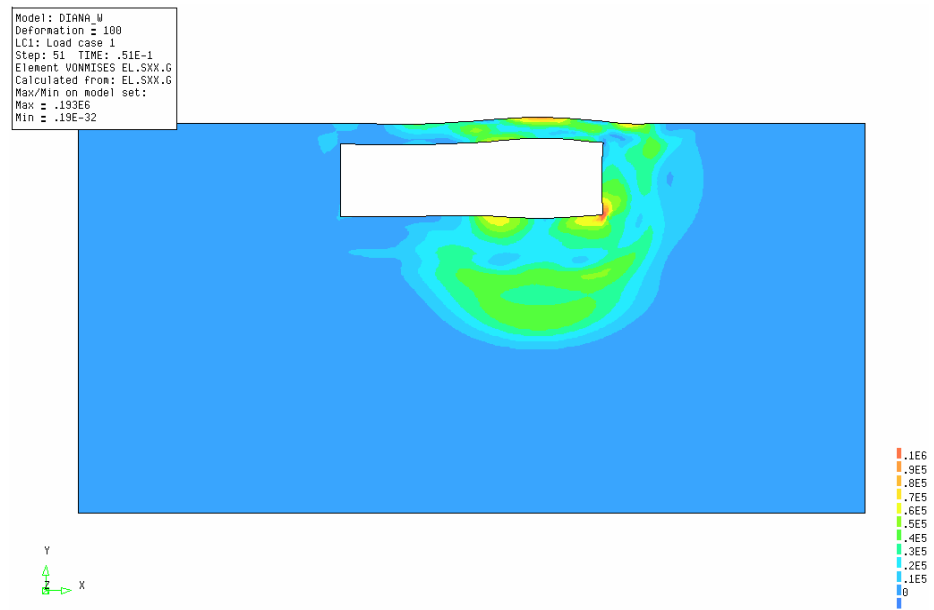


Figure 78 Von Mises stresses in soil at 51 ms

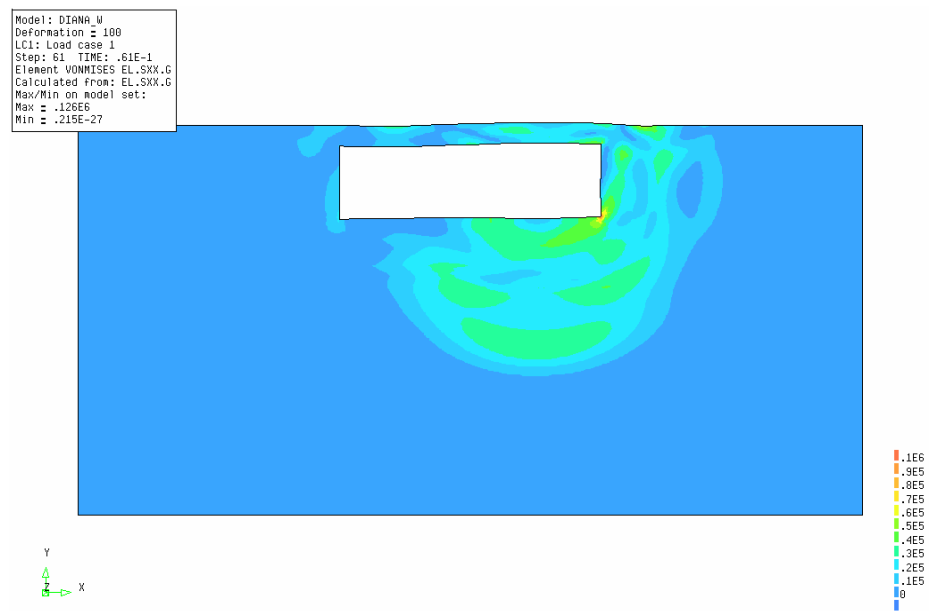


Figure 79 Von Mises stresses in soil at 61 ms

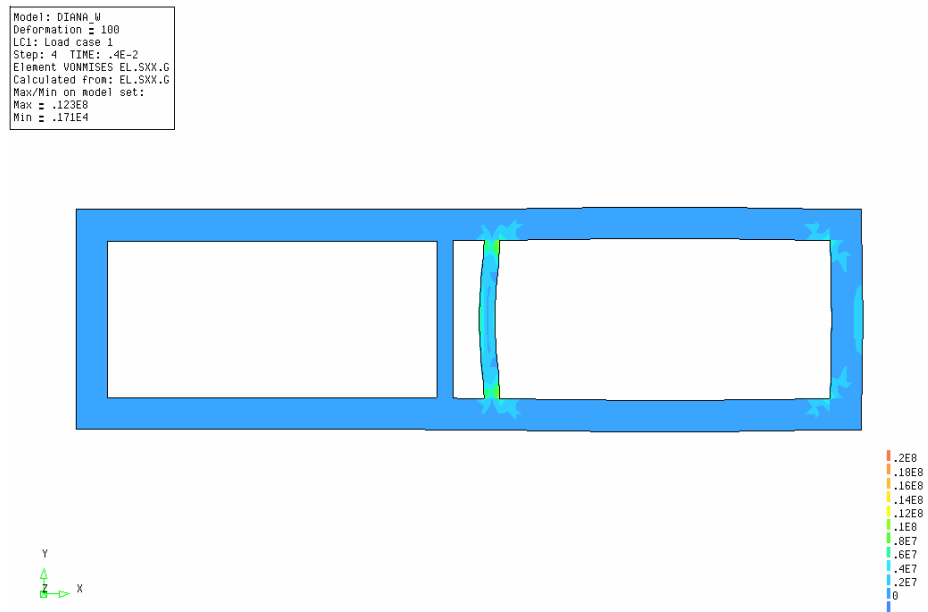


Figure 80 Von Mises stresses in tunnel at 4 ms

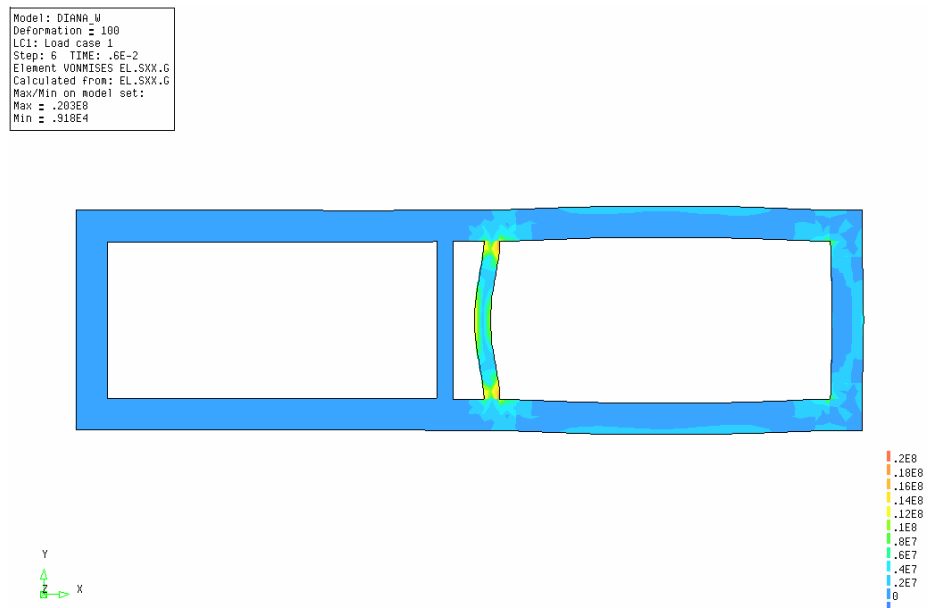


Figure 81 Von Mises stresses in tunnel at 6 ms

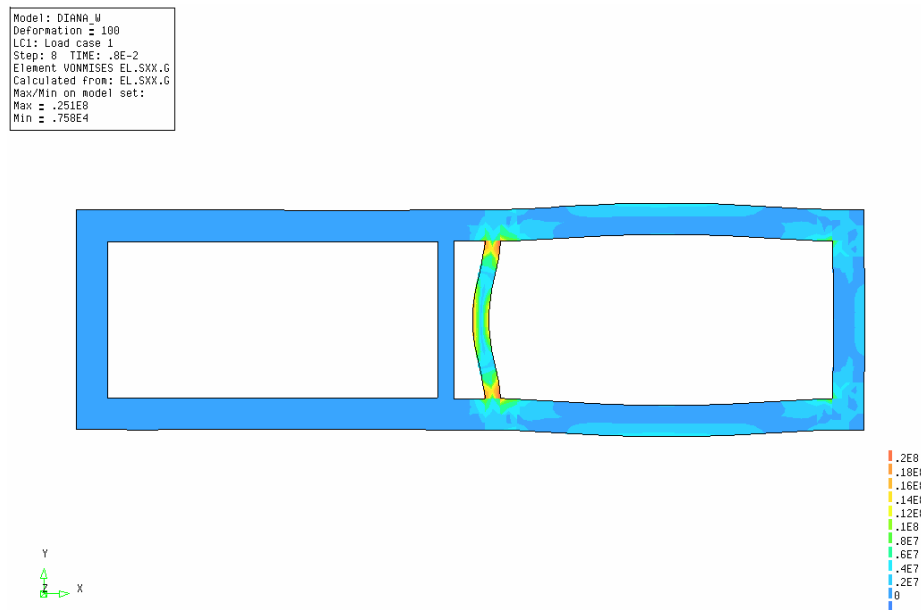


Figure 82 Von Mises stresses in tunnel at 8 ms

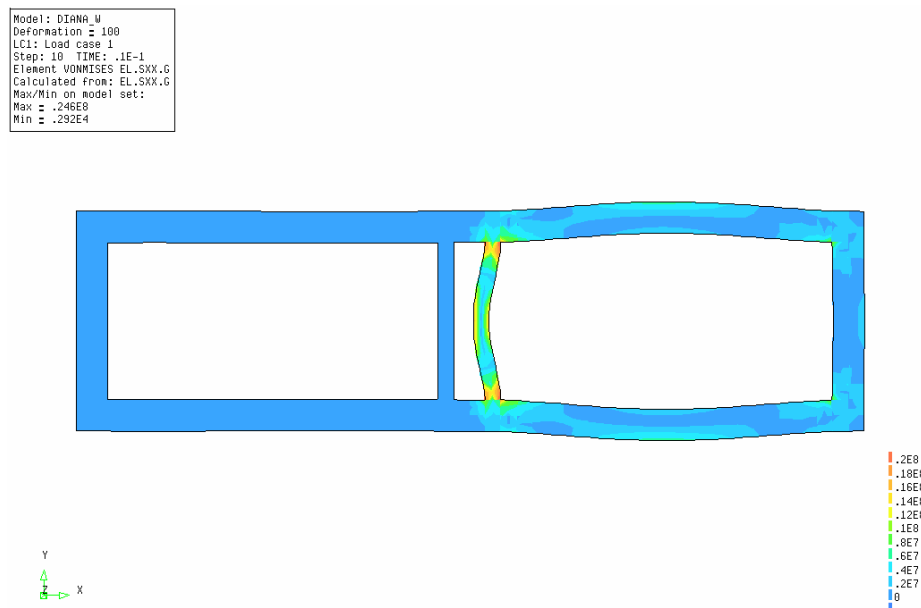


Figure 83 Von Mises stresses in tunnel at 10 ms

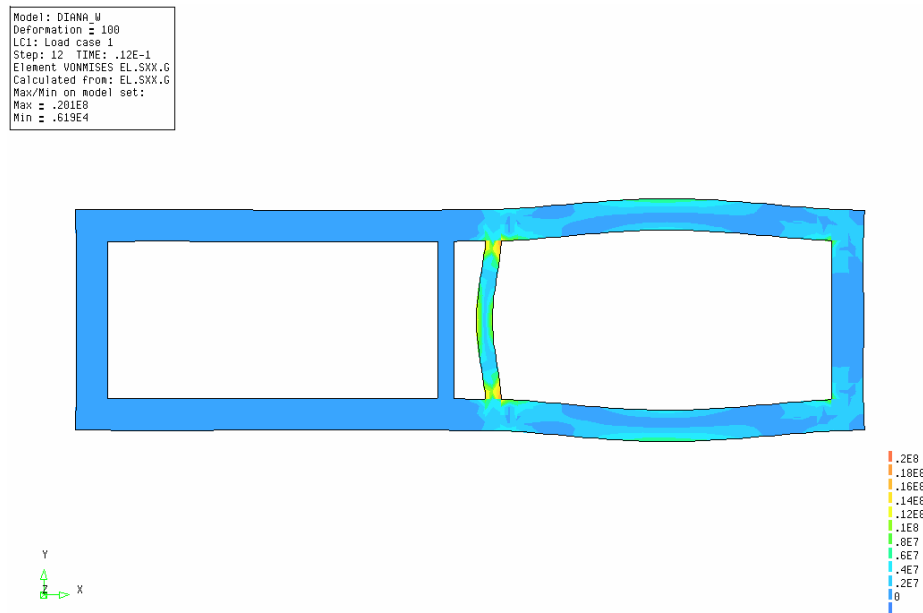


Figure 84 Von Mises stresses in tunnel at 12 ms

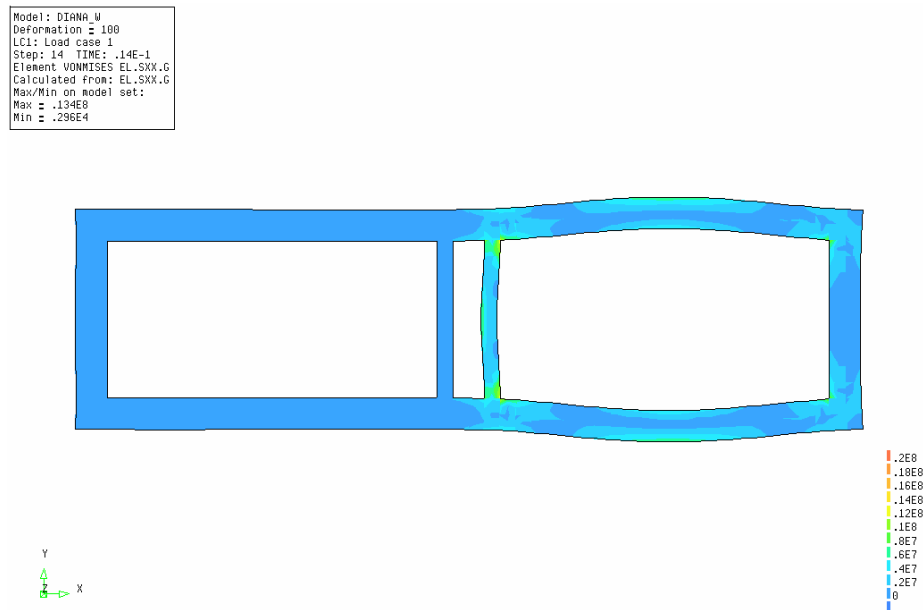


Figure 85 Von Mises stresses in tunnel at 14 ms

Table 18 Stresses and section forces at three different sections of the right separation wall at $t=8$ ms.

	σ_{yy}		σ_{xx}		Moment	Shear
	N/mm ²		N/mm ²		kNm/m	kN/m
top	-19.4	22.7	-1.7	2.5	1263	240
middle	19.6	-16.2	0.02	0.3	-1074	96
bottom	-19.4	22.7	-1.7	2.5	1263	240

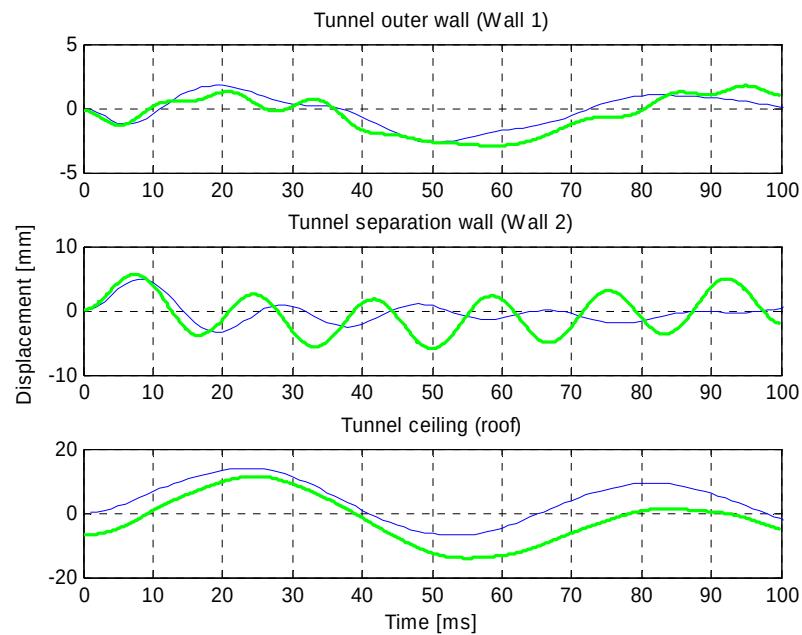


Figure 86 Comparison of tunnel response in 2D membrane and beam simulations without soil (blue thin line and green thick line respectively). For the purpose of the comparison, the sign for deflections of the walls in the simulation with the soil has been reversed. Results from the linear analysis of the Caland tunnel (id 01) are used.

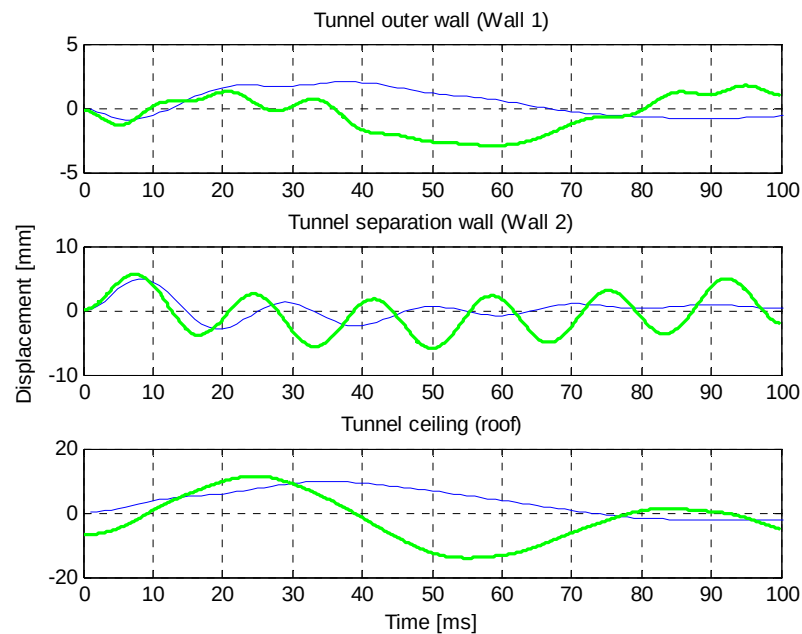


Figure 87 Comparison of the calculated tunnel response using in simulations with and without soil (blue thin line and green thick line respectively). For the purpose of the comparison, the sign for deflections of the walls in the simulation with the soil has been reversed. Results from the linear analysis of the Caland tunnel (id 01) are used.

H.5 Discussion of results

Summary of the results

- The maximum displacement at surface level is 9 mm. This is equal to the maximum displacement of the tunnel ceiling;
- The maximum displacement of right separation wall is 5 mm;
- Inner and outer walls have a higher frequency of vibration than the ceiling and floor (50 Hz for separation wall and 10 Hz for ceiling);
- At onset of rebound (that is, at the maximum displacement amplitude of ceiling and upper soil layer at 30 ms) there are tensile stresses in the soil at the surface and at the soil / tunnel ceiling interface. These stresses are about 150 kPa. As a result the upper layer of soil will separate from the tunnel.
- At the bottom right corner of the soil / tunnel interface a stress concentration appears. At 40 ms the maximum tensile stress in this point is 0.2 N/mm^2 .
- At 8 ms, stress in the right separation wall reaches a maximum. The stresses are quite high and could lead to failure of the wall: the bending stress is $\sim 21 \text{ N/mm}^2$. The maximum shear stress is 2.5 N/mm^2 .

Comparison of the 2 membrane -model (with and without soil) and the beam model

For studying the effect of the soil and the effect of using a membrane model, the deformations of the following three analyses are compared in Figure 86 and Figure 87:

- beam model (corresponding analyses id01 to the chapter 3)
- 2D membrane model, including the soil
- 2D membrane model, without taking into account the soil

From Figure 86 it appears that deflections calculated with the membrane model and the beam model correspond quite well. The main differences are explained by the damping that is taken into account in the membrane model. Moreover a shift of the roof deflection is noticed, that can be explained by the fact that the weight of the structure is not taken into account. Because linear elastic material behaviour is assumed, the (static) effect of the weight may be superposed to the calculated deflections.

Modelling the soil around the tunnel leads to an increase of the vibration period of the tunnel structural components (walls and roof/ceiling). This can be seen in Figure 87. The response is lower for all components. The roof exhibits the largest reduction in displacement amplitude as shown in Table 19.

Table 19 Calculated displacements.

component	model	maximum (mm)	minimum (mm)
roof	beam	11	-14
	membrane	10	-2
wall 1	beam	1	-3
	membrane	+2	-1
wall 2	beam	5.5	-5.5
	membrane	5	-3

As for the forces that occur in the tunnel components a reduction takes place when the soil is considered. The maximum bending moment occurs at the inner wall junctions (about 1.3 MNm/m). The maximum bending moment in the analysis without soil is about 1.8~2.3 MNm/m, see Figure 40.

It seems that not taking the soil (and damping) into consideration leads to an overestimation of the tunnel response and forces. This overestimation is about a factor 2. Of course this is valid only for the present case and may vary according to soil stiffness and explosion type.

Limitations of present analysis:

- Tunnel is modelled with a very rough mesh: stresses distribution in the tunnel is not very accurate;
- High frequency components above 10 Hz are present in the model, for example: the tunnel walls. Because damping increases above 10 Hz, the reaction of these components could be underestimated in the model. The behaviour > 10 Hz could be checked through an analysis with different Rayleigh damping parameters;
- High frequency components > 10 Hz in the loading are also present, even though in not the same intensity as low frequency components.
- The soil is modelled as a single phase material: in fact the soil is completely saturated and the tunnel is situated below a canal, which is not modelled. The effects of a hydraulic pressure have therefore not been taken into account. The two phase behaviour of the soil could lead to a substantially different soil response. It is expected that these effects are studied later on in the project;
- Deviatoric component of the stress in the soil would be interesting to check for possible soil failure mechanisms.