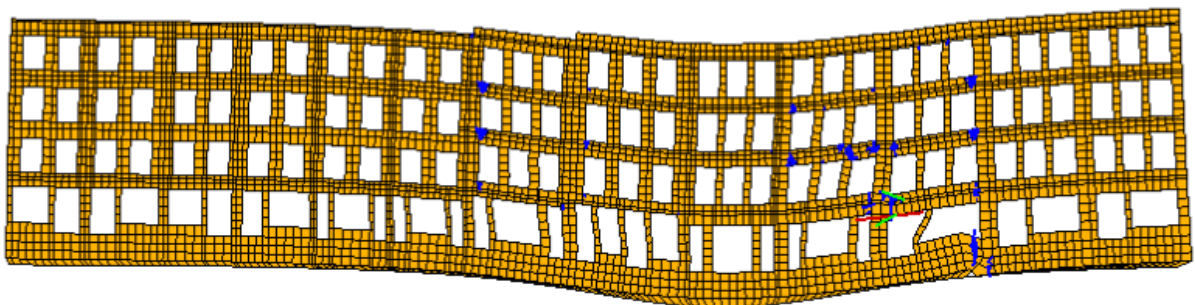


Tunnel induced settlement damage

-A case study to improve damage prediction for facades-



 **TU Delft**

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 **Witteveen+Bos**

Tunnel Induced Settlements

- A case study to improve damage prediction for facades-

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I. Preface

This research project is carried out within the frame work of the final course of the Master of Science program of Civil Engineering at Delft University of Technology. The project took place under the supervision of the Section of Structural Mechanics in cooperation with the engineering company Witteveen+Bos Consulting Engineers.

I'm grateful that Witteveen+Bos provided me the chance to study a recent case, giving me a unique chance to use field measurements to evaluate the available theory. This case study gave me the opportunity to apply the mostly theoretical education to an actual practical problem. Besides that, working within an engineering company gave me a good idea about the daily work within the company, which was a nice bonus to the research.

I would like to thank my graduation committee for the effort they put in my thesis; their positive and useful comments helped me a lot to get to the final result of this thesis. My special thanks go out to Richard Roggeveld, who was always there to help me out whenever I ran into a problem during this thesis. Also I would like to thank Giorgia Giardina and Max Hendriks, who were always ready to discuss the progress of the thesis with me. Furthermore I would like to thank Nicolaas van Empel as well for his useful thoughts and input in my thesis.

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II. Summary

In this thesis the building response due to tunnel induced settlements is studied. A case study is done to examine the response of facades in the Daniel Stalpertstraat due to settlements caused by the tunnelling process. This field data is used to find a calibrated 2D numerical model of the facades.

The calibrated numerical model is then subjected to larger settlements to obtain the behaviour of the facades at large settlements. Using linear and *nonlinear* analyses of the numerical model it is evaluated how accurate and how conservative 4 damage prediction models are. The models used are:

- Two LTSM models, one with $E/G=2.6$ and one with $E/G=12.5$.
- Two models based on conventional beam theory: portal frame model and Forget-Me-Not model.

With linear analyses it is checked how reliable the methods are in terms of strains and deformation under the imposed settlements. With *nonlinear* analyses the conservativeness of each method in terms of damage is evaluated.

The LTSM with $E/G=12.5$ gives the best results according to linear numerical analyses results. The Forget-Me-Not model shows the best results according to the *nonlinear* analyses results.

III. Extended summary

Due to lack of space in urban areas, more underground structures are being built nowadays. Building underground structures in urban areas brings extra risks; soil settlements caused by construction works can affect nearby buildings. Building damage could result in repair costs and puts the progress of the building process at risk. The risk of damage caused by underground structures is assessed in a Settlement Risk Assessment (SRA). In this study a damage prediction is made for buildings that are subjected to underground construction activities. Based on the damage level predicted, it is determined whether and where measures have to be taken to reduce the risk of damage. A reliable damage prediction model is therefore important in the SRA. The method needs to be conservative enough to be sure no harm is done to the buildings; on the other hand a too conservative method would unnecessarily lead to expensive measures to reduce risks.

In this project the aim lies on the damage prediction for facades of masonry houses due to soil movements caused by a tunnel boring process.

One of the methods often used for damage prediction to masonry buildings is the Limiting Tensile Strain Method (LTSM). The LTSM models a masonry building as a weightless, isotropic, linear-elastic, rectangular beam on 2 supports, with a height H , length L and a material parameter E/G . Although the LTSM is an easy method to use, it has its limitations. For façade like structures for instance the perforation of the wall causes a reduced stiffness and results in different contributions of bending and shear deflection. The openings in the façade also are weak spots; the stress concentrates at the corners of the windows. These peak stresses can cause a façade to crack earlier than is predicted by the LTSM. Nonlinear effects are neglected in the LTSM as well. The only way to take into account this influence of perforations is adjusting the E/G ratio in the method. The applicability of the LTSM as a damage prediction method in the case of facades under a transversal settlement trough is studied in this project. Alternative damage prediction methods are examined for their applicability as well.

A part of the new metro line in Amsterdam, the North/Southline, is used as a case study. In the district “De Pijp” the Tunnel Boring Machine (TBM) passes right underneath 5 blocks of houses. In the Daniel Stalpertstraat the tunnel alignment runs perpendicular to the alignment of the facades of the houses. This provides the opportunity to study the response of facades subjected to a transversal settlement trough. The vertical soil surface movements and the horizontal and vertical movements of the facades are monitored extensively. The measurements in the first 8 weeks after passing of the TBM are analysed.

Reliable results were found for the vertical soil and façade movements. It appeared that the measurements of the horizontal movements of the facades were within the margin of error of the measurement system; the exact values are less accurate and are considered not to be reliable enough for further analyses. The soil displacements and the consequent building response did not cause any cracks in the facades.

The field data is used for the calibration of a numerical model of the facades. With the help of the field data the masonry Young’s modulus and the stiffness of the foundation can be determined:

- Linear analyses are used to match the building movement measured at site to the movements of the numerical model.
- *Nonlinear* analyses are used to obtain whether the numerical model stays uncracked under the imposed settlements.

The geometry of the facades of the Daniel Stalpertstraat was measured at site to obtain a reliable representation of the geometry of the houses in the numerical model. A numerical 2D plane stress model is used for verification of the field data. A semi-coupled model is used; interface elements take into account the stiffness of the foundation piles.

Using linear analyses several combinations of the masonry Young's modulus (E_m) and normal interface stiffness (k_n) of the interface were found that match the numerical results to the field data. Because the field data of the horizontal movements of the facades were less accurate, no tangential interface stiffness could be derived. The tangential interface stiffness is based on the work of other researchers.

The combinations of model properties found in the linear analyses are checked for cracking with a *nonlinear* masonry model. Since the tensile strength is unknown, it was assumed to be 0.3 N/mm^2 . The choice of this tensile strength is based on literature and the experience of Witteveen+Bos. The Young's modulus was found to be 1000 N/mm^2 and the normal interface stiffness $1.2 \cdot 10^7 \text{ N/m}^3$. It is emphasized that the assumption of the tensile strength influences the range of possible of the masonry Young's moduli.

The tunnel induced settlements in the case study were small; too small to cause any damage. In order to obtain the behaviour of the facades under increased settlements, the imposed settlements in the numerical model have to be increased. Consequently, in the remaining part of this study, three reference analysis results are used:

- The results from the calibrated linear model described above.
- The extrapolated results from this model with an increased settlement profile.
- The extrapolated results from a *nonlinear* model with an increased settlement profile. It is assumed that these results reflect the behaviour of the facades under large soil settlements

The results of the three analyses are compared to the results of 4 models for damage prediction. Two of them are LTSM models:

1. Standard LTSM model (E/G ratio of 2.6). The settlement profile applied in the numerical model is used for calculations.
2. LTSM model with adjusted LTSM ratio (E/G ratio of 12.5). The settlement profile applied in the numerical model is used for calculations.

The other two are treated in the linear analyses.

The results of the aforementioned linear and *nonlinear* analysis results will be compared to the results of the damage prediction methods as follows:

- ✓ Linear model: the strains and deformations found in the linear numerical analysis results are compared to the strains and deformations calculated in the damage prediction models. This is done because the damage prediction models are linear models, calculating strains and deformations using linear elasticity. The accuracy of the damage prediction models in terms of strains and deformations can be determined with this comparison.
- ✓ *Nonlinear* model: the maximum crack width found in the *nonlinear* analysis results compared to the crack width range determined by translating the calculated strains from the damage prediction models to crack widths. The *nonlinear* numerical model takes into account nonlinear effects (post cracking behaviour), while this is not taken in the damage prediction methods. The conservativeness of the damage prediction models can be evaluated.

First the linear analysis results are considered. The results showed that, in order to explain the behaviour of the linear numerical model, a distinction has to be made between the *local* and *global* behaviour of the facades:

- *Global* behaviour: the behaviour of the facades all together, as if it is a beam. This is also the approach used in the LTSM. The deformation in terms of shearing and bending are determined. The results show that the curvature of the facades is minimal, the shear distortion is large; shear distortion is the dominant deformation mode. This behaviour does not change under increased settlements. The global shear strains are extracted from the neutral axis of the facades, the global bending strains from the bottom of the facades in the sagging zone. It was found that the maximum global shear strains are larger than the

maximum global bending strains, which is a consequence of the high shear distortion of the facades. The strains found in the global behaviour however are not the maximum strains in the numerical model. The results of the global strains and deformation are compared to the results of the two LTSM models, since this model also considers the strains at the mentioned locations.

- *Local* behaviour: the behaviour of an individual façade. The individual façade deforms like a portal frame under shear. The bending of individual parts of the façade introduces local bending strains that are higher than local shear strains. The local behaviour does provide the maximum strain in the model.

It is not possible to simulate the *local* behaviour with the LTSM (*global* behaviour); two new methods are introduced, based on conventional beam theory that could represent the local behaviour:

3. *Portal frame model*: portal frame model with stiffness's of individual parts of the considered façade, the applied shear distortion is the maximum first derivative of the settlement profile. The strains in the beams of this portal frame model are compared to the local strains in the beams of the numerical façade model. The major disadvantage of this model is that the columns of the portal frame method provide the maximum strains in the portal frame model, while this is not obtained in the local behaviour of the numerical model.
4. *Forget-me-not model*: the maximum first derivative of the settlement profile is applied again. With this model the columns are assumed to be infinitely stiff; this overcomes the problem of the columns that are responsible for the maximum strains. The maximum local strains of the numerical model are compared to strains calculated in the forget me not model.

In the *nonlinear* analysis results it was obtained that the *global* behaviour of the facades does not change due to cracking, shear distortion remains the dominant deformation mode. The model starts to crack at the corners of openings in the façade with the highest shear distortion. The cracks are initialised by the bending strains caused by the *local* behaviour of the numerical model.

The strains found in the 4 damage prediction models are translated to a crack width. This crack width is compared to the width of the largest crack found in the numerical model.

The conclusion from the comparison of between the numerical results and the damage prediction models are:

1. Standard LTSM model: does not show good comparison with the deformation of the linear numerical model. It overestimates the curvature and underestimates the shear distortion. This model overestimates the bending strains and underestimates the shear strain; bending strains are maximum in this method, while this was not obtained in the *global* behaviour of the linear numerical model. The method, if compared with the *nonlinear* numerical model, is not conservative at large soil settlements.
2. LTSM model with adjusted LTSM ratio. This model shows similar results in terms of deformation if compared to the *global* behaviour of the linear numerical model. The calculation of the shear strains is quite accurate, the bending strains are underestimated. The method, if compared with the *nonlinear* numerical model, is not conservative at large soil settlements.
3. Portal frame model. The portal frame method predicts that that the columns are providing the maximum strains, while this is not obtained in linear numerical model. If the columns are neglected, the results in terms of strains are not accurate either, although the calculated bending strains are larger than shear strains. This was also obtained in *local* behaviour of the linear numerical model. At large soil settlements the method is not conservative if compared with the *nonlinear* numerical model.
4. Forget-Me-Not model: The method does calculate that bending strains are larger than the shear strain as obtained in the *local* behaviour of the numerical model, but bending strains are overestimated. The model however show reliable results if compared with the *nonlinear* numerical model.

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IV. List of symbols

A	cross section area	(m ²)
c	cohesion	(-)
D	tunnel diameter	(m)
E	Young's modulus	(N/mm ²)
f_t	tensile strength	(N/mm ²)
G	shear modulus	(N/mm ²)
H	height	(m)
I	moment of inertia	(m ⁴)
i_x	inflection point distance	(m)
K	dimensionless trough width parameter	(-)
k_n	normal stiffness interface elements	(N/m ³)
k_s	tangential stiffness interface elements	(N/m ³)
L	length	(m)
M	bending moment	(Nm)
N	normal force	(N)
S_h	horizontal displacement	(m)
S_v	vertical settlement	(m)
V	shear force	(N)
V_L	volume loss ratio	(-)
V_S	volume of the settlement trough	(m ²)
W	section modulus	(m ³)
w_{crack}	crack width	(mm)
z	depth beneath ground surface	(m)
z_0	tunnel depth	(m)
α	angular strain	(-)
α_s	form factor	(-)
β	relative rotation	(1/m)
γ	shear distortion	(-)
Δ	relative deflection	(m)
ε_b	bending strain	(-)
ε_d	shear (diagonal) strain	(-)
ε_h	horizontal strain	(-)
ε_t	tensile strain	(-)
θ	rotation or slope	(1/m)
κ	curvature	(1/m)
ν	Poisson ratio	(-)
ρ	density	(kg/m ³)
τ	shear stress	(N/mm ²)
ω	tilt	(1/m)
A	cross section area	(m ²)

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1. Introduction and scope

1.1 Background

A new metro line is being built in Amsterdam; the North/Southline. The greatest part of the metro line is constructed using a Tunnel Boring Machine. The tunnel alignment of the North/Southline can be found in Figure 1.

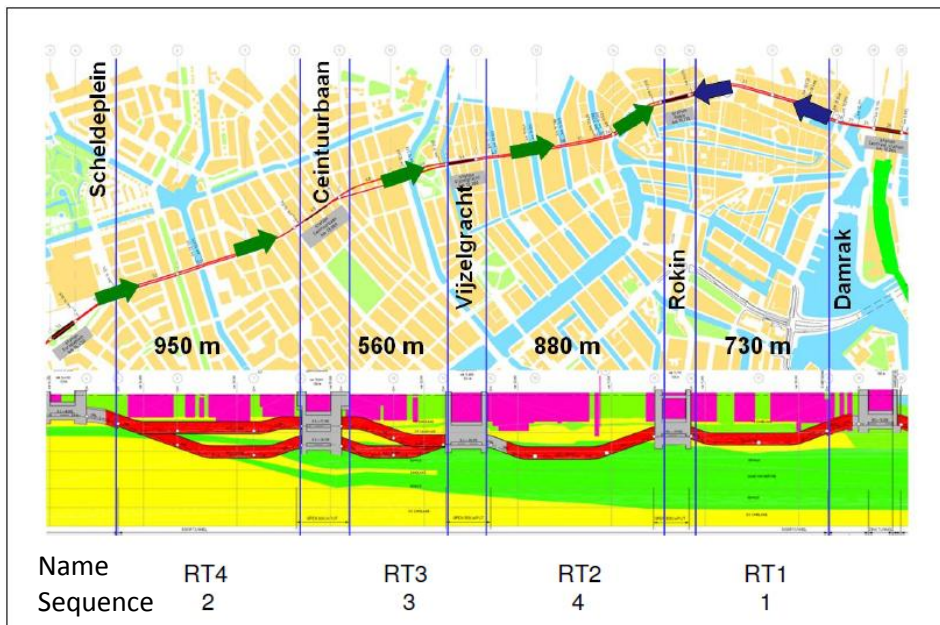


Figure 1: Tunnel alignment and different sequences of the North/Southline

In November of 2011 a special part of the tunnel of the North/Southline was constructed; the part underneath the district “De Pijp”. This part is called Running Tunnel 3, RT3 in Figure 1. The tunnel runs directly underneath the houses of “De Pijp”, perpendicular to the alignment of the facades. Therefore it is also part with a higher risk; tunnel induced settlements could cause damage to the buildings. In this thesis one part of “De Pijp” is studied; the Daniel Stalpertstraat.

In order to reduce the risks of damage to the buildings above or near the tunnel, a Settlement Risk Assessment (SRA) was performed for every part of the North/Southline where tunnel induced settlements can cause building damage. By evaluating the vulnerability of the affected houses to damage and estimating the soil settlements to be expected, the risk of damage to houses was determined. Where necessary, measures were taken to reduce the risk of damage to the houses, like mitigation measures during the boring process or installing a new foundation.

The soil and building movements were extensively monitored during the boring process. If the soil or building movements become too large, measures need to be taken to decrease the movements. The measured building and soil settlements are collected in the Geological Information System (GIS). This data is valuable and can be used to gain a better insight in the building response due to tunnel induced settlements.

A common method to predict damage to buildings in the preliminary design stage is the Limiting Tensile Stain Method (LTSM). This method assumes a building to be a linear solid beam on 2 supports. Using the relative displacement of the beam, bending and shear strains can be determined. The magnitude of these strains can be translated to an indication of damage to the building. If this damage indication is above a certain level, measures need to be taken to reduce the risk of damage.

1.2 Scope of the project

In this thesis the behaviour of facades due to tunnel induced settlements is examined. In the case of facades under transversal settlements the LTSM is often applied, in which the façade is modelled as a solid beam. But, due to the many perforations of the facades, it is not expected to act like a solid beam because of the reduced shear stiffness of the facade. The perforations of the façade cause decreased shear stiffness, making the façade deform more in shear deformation and reducing the bending deformation. In the case of a solid beam, as is used in the LTSM, this bending does play an important role.

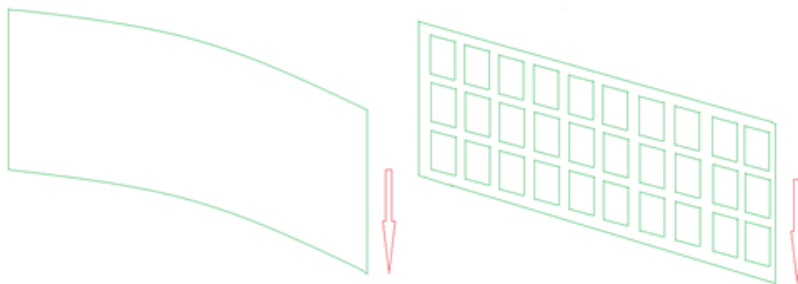


Figure 2: Bending of a beam and shearing of a facade

These different deformation modes give rise to different bending and shear strain contributions. Furthermore the openings cause peak strains near the corners and are therefore vulnerable parts of the façade, which is not taken into account in the LTSM. The case study of the Daniel Stalpertstraat should provide more insight in the façade response to tunnel induced settlements. This information can then be used to improve the damage prediction in case of facades.

The main goal of this case study can be stated:

“Examine the behaviour of a façade due to tunnel induced settlements and use the results for improvement of the damage prediction for facades”

The thesis is built up in 4 parts:

1. The first step is *analysing the monitoring data*. Because of the extensive monitoring of the soil and buildings during the tunnelling process the soil settlements and the according building response can be examined. The data of a time span of 8 weeks is examined to determine the building response over time;
2. The second step is to make a *2D numerical model* of the facades. The measured settlement trough can be applied to the model. By adapting the material and interface properties, it should be possible to match the response of the 2D building model to the monitored façade behaviour. This step should result applicable material and interface properties. Using the field data also increased settlements troughs can be determined;
3. Third step is to perform *linear analyses of the increased imposed settlements*. The maximum strains and the shear and bending deformation of the numerical model is examined under increased settlements. In order to explain the behaviour of the numerical model a distinction is made between the local and global behaviour of the facades. The results of the linear numerical model are compared to the results of 4 damage prediction methods. The applicability of each method in the case of the facades in the Daniel Stalpertstraat is determined.
4. The fourth step is to perform *nonlinear analyses of the increased imposed settlement*. The behaviour of the numerical model under larger settlements might change because of crack

initiation. The crack width occurring in the facades under the imposed settlements are determined. The results are compared to the damage classification by the damage prediction methods from step 3. The applicability of each damage prediction method is determined for the case of the nonlinear numerical model.

Although the subject is at the interface of Structural and Geotechnical Engineering, the emphasis of the thesis will lie on the structural behaviour of masonry façade due to imposed settlements.

1.3 Outline report

This report will begin with a sort summary of the literature that has been used during the thesis (Chapter 2). It is assumed that the readers already are familiar with the theory used for tunnel induced settlement damage. Therefore the summary will contain only the headlines of the theory; for further elaboration reference is made to literature.

In Chapter 3 the results of the case study of the Daniel Stalpertstraat will be described and a comparison is made to other streets in “De Pijp”. The data found in this case study will be used for further analyses

In Chapter 4 the numerical model of the Daniel Stalpertstraat is described. Using the field data applicable model parameters will be determined which results in a calibrated numerical 2D model. The field data is used to determine increased settlement profiles.

In Chapter 5 the behaviour of the calibrated linear numerical model will be examined for its behaviour under increased imposed settlements. The strains and deformations found in these analyses will be compared to the results of 4 damage prediction methods. A separation is made between the local and global behaviour. The accuracy of each damage prediction model according to the linear analyses results will be evaluated.

In Chapter 6 a nonlinear numerical model is used to examine its behaviour under large imposed settlements. The crack widths in the numerical model will be determined. The strains obtained in the damage prediction models will be translated to crack widths. By comparing these results to the results of the nonlinear numerical analyses, the conservativeness of each model is evaluated.

Finally conclusion and recommendation from the thesis will be summarized.

2. Theoretical background.

As was mentioned before, the tunnelling process introduces settlements which may affect building above or near the tunnel. Many researchers have studied the building response due to tunnel induced settlements. In this chapter a brief overview is given of the theory used for this thesis; for more information reference is made to the literature, which can be found in the literature list in the back of this report.

First the different mechanisms responsible for tunnel induced settlements are treated. In the next paragraph the shape of the settlement trough introduced by tunnelling, the greenfield settlement profile, will be described. Throughout the report the formulas in this paragraph, introduced by Peck, will be used to determine settlement profiles. The characteristics of these greenfield settlements are used in empirical methods to predict settlement induced damage to building. The empirical methods are treated in Chapter 2.3.

A more common used method to translate the soil settlement to building damage is the Limiting Tensile Strain Method (LTSM). This is a major topic in this thesis and a description of this method can be found in Paragraph 2.4.

In the last part of this chapter a short summary of the numerical methods used in the past will be treated. The soil settlements and building behaviour can be simulated with a numerical model.

2.1 Tunnel induced settlements

Amsterdam has soft soil conditions; the construction of a tunnel in soft soil conditions will lead to ground movements. The magnitude of the ground movements depends on several factors. One factor is the type of boring machine used. For the North/Southline a closed tunnel boring machine is used with a so called slurry shield to maintain the right pressure in front of the cutter head. The cutter head is rotating to cut the soil, and is advanced using jacks that are placed on the face of the already existing tunnel lining. The soil in front of the cutter head wants to move towards the cutter head; this is called face loss. The slurry shield, or Earth Pressure Balance (EPB), provided with the right pressure can reduce this face loss, although total prevention cannot be achieved. The amount of face loss depends on the applied pressure and the skills of the boring team.

The cutter head of the TBM is slightly bigger than the rest of the TBM. This extra space is needed to manoeuvre the TBM to build for instance the bends in the tunnel alignment. The gap created between the cutter head and the rest of the shield gives rise to radial losses, which cause additional settlements. Behind the shield the concrete lining is installed. The gap between the tunnel lining and the soil, the tail void, is even bigger (up to 0.2m) and causes settlements as well. This gap is directly injected with grout under pressure, under the right grout pressure the settlement can be kept within limits. The tail void loss can also be introduced by an insufficient amount of injected grout. The different losses during the boring process are schematised in Figure 3

The concrete lining of the tunnel itself can deform (shrink) under the pressure of the soil. The settlement caused by this shrinkage is generally very small.

Summed, the total settlement of the soil has three causes:

- Face loss;
- Radial loss;
- Tail void loss;

These losses all together, combined with the built up of soil layers, determine the tunnel induced soil settlements.

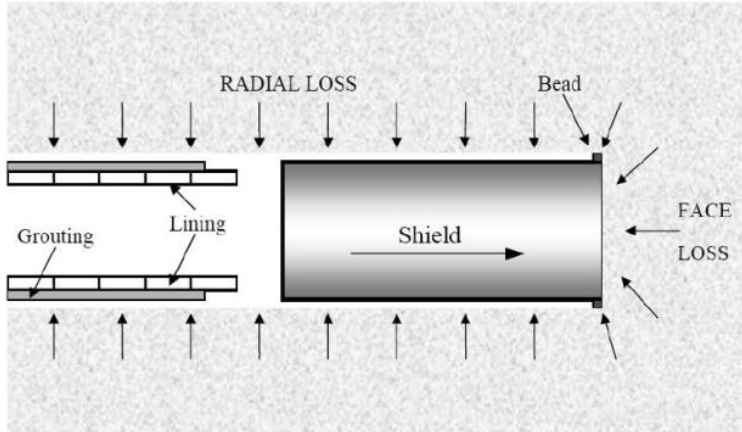


Figure 3: Schematic cross section of a TBM, Bloodworth (2002)

2.2 Tunnel induced settlement prediction

As described in Paragraph 2.1, the construction of tunnels introduces soil settlements. In practise a widely used method for the prediction of these tunnel induced settlements was described by Peck (1969), which will be treated in this paragraph. A distinction is made between settlements in transversal and longitudinal direction.

2.2.1 Settlements in transversal direction

Settlements in transversal direction, also referred to as the transversal settlement trough, are settlement perpendicular to the tunnel alignment. Soil movements in transversal direction can be subdivided into vertical and horizontal ground movements. For the prediction of tunnel induced settlements greenfield conditions are used, or the settlements of the soil in absence of any structure. Current greenfield settlement prediction models are based on the work of Peck (1969). Peck has created a semi-empirical method for predicting the soil movements in longitudinal and transversal direction, based on the Gauss curve. Emphasized is that greenfield conditions do not take into account soil-structure interaction.

Vertical soil displacements

Peck introduced a formula for the prediction of short term transversal settlements based on the on data from 20 historical cases. Using these cases he was able to come up with a formula based on Gauss error function that was representative for these cases. The vertical displacements in transversal direction can be described by the following formula:

$$S_v(x) = S_{v,max} e^{-\frac{x^2}{2i_x^2}} \quad (1)$$

In which:

- $S_{v,max}$ = maximum vertical settlement of the transverse trough at the tunnel centreline
- x = horizontal distance from tunnel centreline
- i_x = horizontal distance between the point of inflection and the tunnel centreline

A sketch of this transversal soil settlement trough can be found in Figure 4.

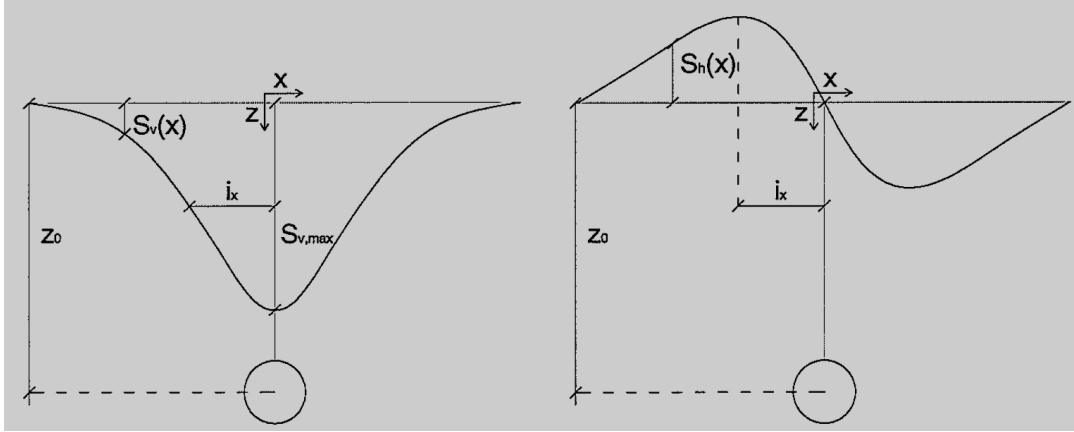


Figure 4: vertical and horizontal soil settlement profiles in transverse direction by Peck, (1969)

As can be seen from the figure, the steepest point of the curve is present at the distance i_x from the tunnel centreline, which is the inflection point of the vertical settlement profile.

The horizontal distance i_x can be determined by the following formula, according to O'Reilly and New (1982):

$$i_x = K * (z - z_0) \quad (2)$$

In which:

- K = dimensionless trough width parameter
- z_0 = tunnel depth
- z = depth beneath ground surface of the considered settlement trough

It is hard to predict a correct K-value because it depends on the type of soil and TBM process parameters such as tail void pressure, front pressure and injection volumes (Netzel 2009). Analyses of field data gives provide best indications of the K-value. The trough width parameter K has been determined by different researches using these field data. For soft soil conditions the K-factor can vary between about 0.4 and 0.7. A common used K-factor is 0.5. Netzel (2009) analysed field data from 3 tunnels built in Holland and determined the K-factor for Dutch projects to be 0.35.

A good indicator for the maximum settlement, often used in practise, is the volume loss ratio (V_L). The volume loss ratio is the total surface of the transversal settlement trough divided by the tunnel diameter. The total volume of the settlement trough can be found by integrating the settlement profile over the full width of the trough:

$$V_S = \int_{-\infty}^{\infty} S_v(x) dx = \sqrt{2\pi} i_x S_{v,max} \quad (3)$$

The volume loss ratio can be found by dividing this volume loss of the settlement trough by the surface of the tunnel:

$$V_L = \frac{V_S}{\frac{1}{4}\pi D^2} \quad (4)$$

In which

- D = tunnel diameter

Equations 1,2,3 and 4 give a relation between the volume loss ratio and the settlement profile:

$$S_v(x) = \sqrt{\frac{\pi}{2}} \frac{V_L D^2}{4i_x} e^{\frac{-x^2}{2i_x^2}} \quad (5)$$

The volume loss ratio is not at forehand known and differs in every tunnelling project. In practise values are used in the range of 1% to 3%. According to the field data analyses for Dutch cases by Netzel (2009) volume losses of 0.15% up to 1.5% were found in three TBM projects in soft soil in the Netherlands.

Horizontal displacements

Next to the vertical soil displacement, a horizontal movement of the soil occurs as well. O'Reilly et al. (1982) derived an equation for the horizontal displacements in transverse direction under greenfield conditions as follows:

$$S_h(x) = -\frac{x * S_v(x)}{z_0} \quad (6)$$

The horizontal displacements are directed towards the tunnel axis, a plot of the horizontal displacements can be found in Figure 4. The horizontal displacements are validated with field data and are often considered to be conservative.

2.2.2 Settlements in longitudinal direction

The advancing TBM creates a longitudinal settlement trough as well. This longitudinal trough proceeds with the advancing TBM and will have disappeared once the boring process is done. The longitudinal trough however can give rise to building damage as well, depending on the position of the tunnel track relative to the dimensions of the building structure. The longitudinal trough is as well described in horizontal and vertical direction. The settlements in longitudinal direction will not be further elaborated in the main report since they will not be considered in this thesis. An illustration of the longitudinal settlement trough can be found in Figure 5.

The combination of the transversal and longitudinal settlement trough result in a 3D trough which is illustrated in Figure 6.

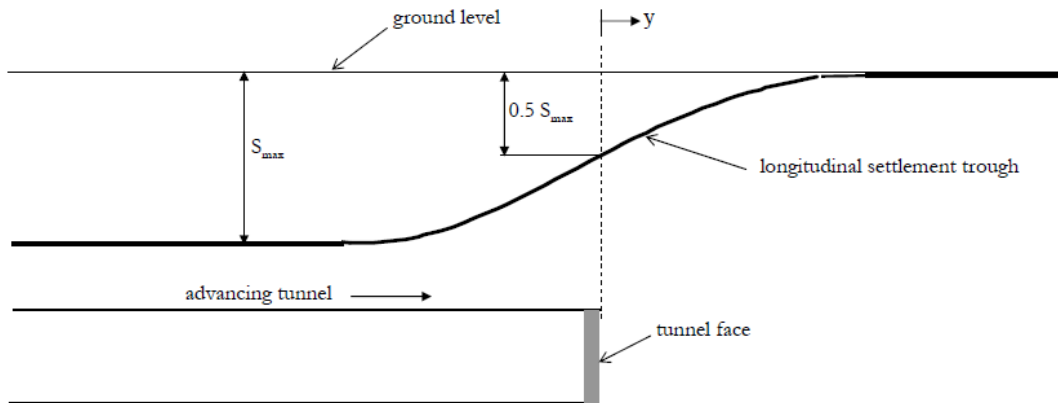


Figure 5: Longitudinal settlement trough, van Abeelen (2009)

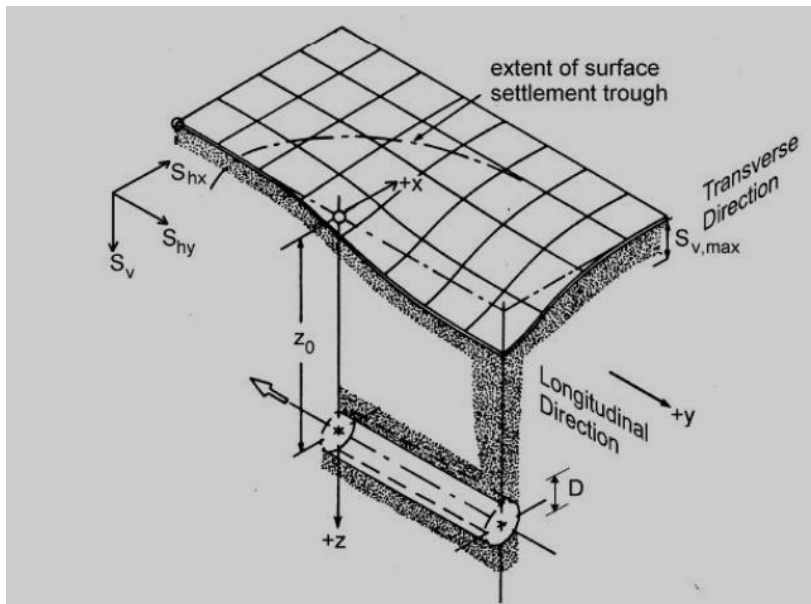


Figure 6: 3D settlement trough profile by Burland et al. (2001)

2.3 Empirical methods for settlement damage prediction

Researchers in the past have studied cases of tunnel induced buildings damage to get a better insight to building response to imposed settlements. From these data they have made recommendations on allowable settlements, which can be used as design guideline. Burland and Wroth (1974) have defined several parameters for building distortions which could lead to damaged structures, as can be seen in Figure 7. The parameters are defined as:

- Settlements (S): vertical movement of the building;
- Differential or relative settlements (δS): difference between 2 settlement values;
- Rotation or slope (θ): change in gradient of a line joining two points;
- Angular strain (α): the change in angle between two straight lines joining two points of the building base;
- Relative deflection (Δ): the displacement of a point relative to a line connecting two reference points on either side
- Deflection ratio (Δ/L): relative deflection divided by the length of the structure;
- Tilt (ω): rigid body rotation of the structure;
- Angular distortion or relative rotation (β): the rotation of the line joining two points relative to the lift;
- Average horizontal strain (ϵ_h): the change in length of corresponding length of the building;

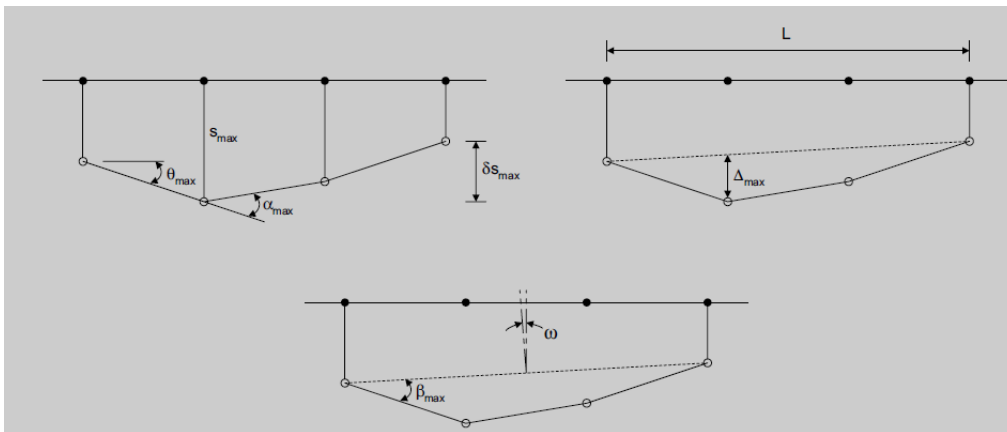


Figure 7: Damage parameters determined by Burland et al. (1974)

It is hard to make a good prediction on the basis of just one parameter, very large datasets have to be analysed. Several researchers did analyse field data and have defined limit values for one or more of these parameters. Limit values have been listed in Table 1 and Table 2. It is emphasized that these values are representative for masonry building on a soil consisting of clay.

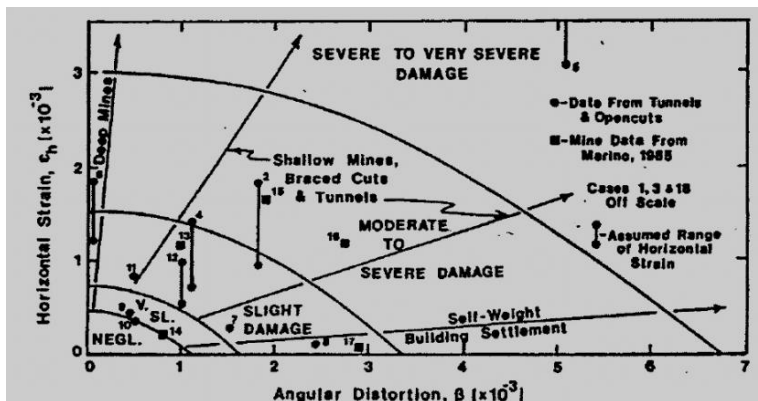
Deformation Parameter	Demand	Source
Settlements	$< 150 \text{ mm}$ $< 80 \text{ mm (L/H} > 2,5)$ $< 100 \text{ mm (L/H} < 1,5)$ $< 100 \text{ mm}$	CUR 162 Polshin and Tokar (1957) Polshin and Tokar (1957) Grant, Cristian & Vanmarcke (1974)
Differential/relative settlements	$< 55 \text{ mm}$ $< 1:300 \text{ or } 1:150$	Grant, Cristian & Vanmarcke (1974) Eurocode 7
Angular distortion/relative rotation	$< 1:300$ $< 1:500$ $< 1:3333 - < 1:2500 \text{ (L/H} < 3)$ $< 1:2000 - < 1:1429 \text{ (L/H} > 5)$	CUR 162 Onderzoek kwaliteit fundering en casco (1992) Polshin and Tokar (1957) Polshin and Tokar (1957)

Table 1: Limit values for different parameters

Deformation Parameter	Negligible	Very slight	Slight	Moderate	Severe	Very severe	Source
Differential/relative settlements	$< 1:500$	1:500 - 1:300	1:500 - 1:300	1:300 - 1:100	1:300 - 1:100	$> 1:100$	CUR 162
Angular distortion/relative rotation	$< 1:500$	1:300 - 1:150	1:300 - 1:150	$> 1:150$			Skempton and MacDonald (1956)
	$< 1:600$			$< 1:300$			De Kock (1982)

Table 2: Limit values for different parameters

Also several design graphs have been developed to determine the amount of damage to a structure due to settlements; they are visualized in Figure 8, Figure 9 and Figure 10 below.

Figure 8: design chart for $L/H=1$ developed by Boscardin et al. (1989)

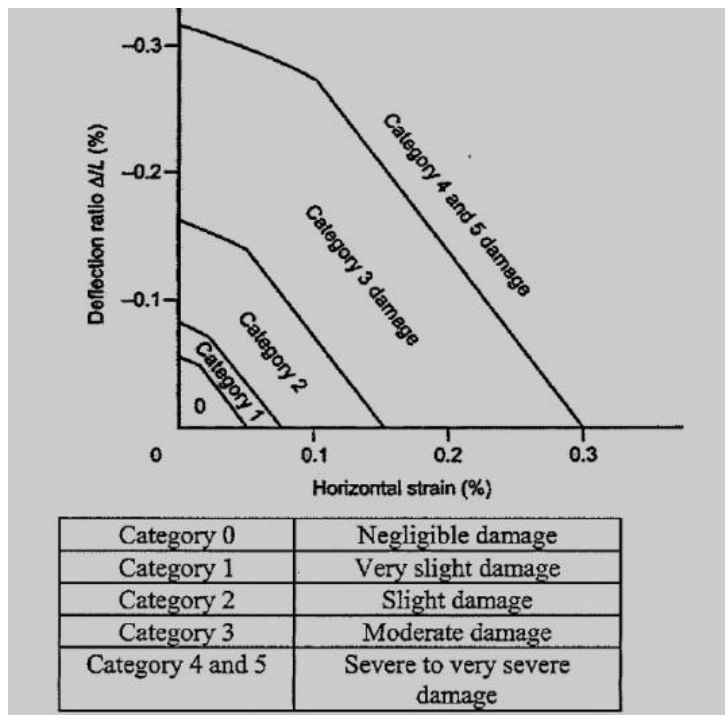


Figure 9: Design chart for $L/H=1$ developed by Burland (1995)

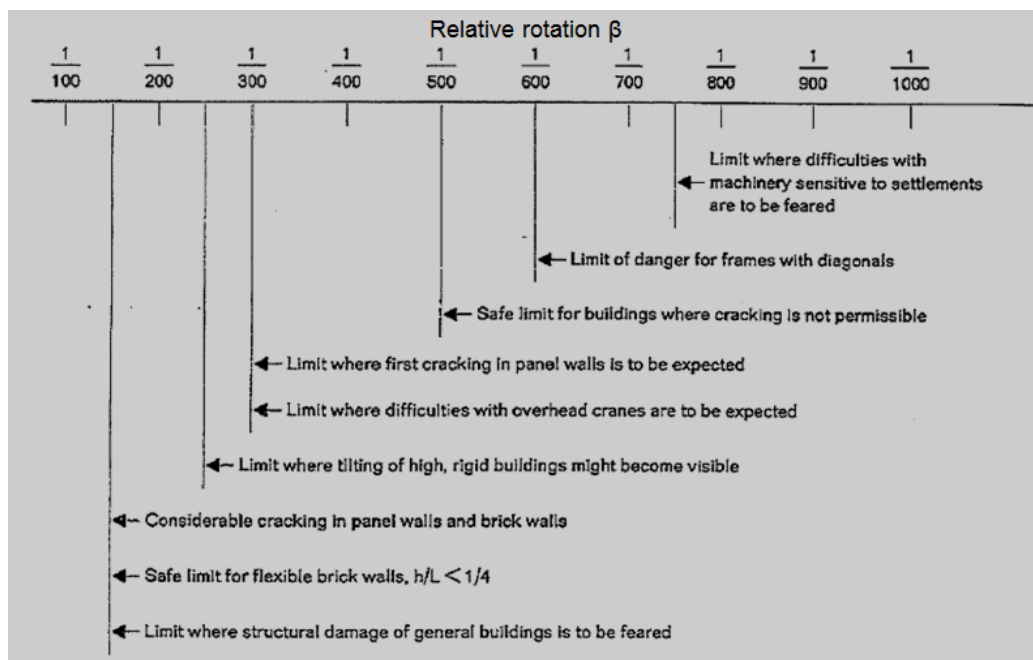


Figure 10: Damage criteria according to Bjerrum (1963)

It can be said that the works of different researches do not always coincide: there is quite a scatter in the presented results. This could be due to the definition of 'structural damage', the applied materials and geometry in the damaged buildings or the differences in soil conditions.

Another value used in practise is the relative displacement between the building and connecting ground surface of the soil. If a building settles more or less than the surrounding ground surface of the building, underground utilities could be harmed (like water pipes, electricity cables, sewerage etc.). A practical relative displacement of 50 mm is often adopted.

Once the greenfield displacement due to tunnelling is known, one should determine from the table or graphs whether a certain settlement is allowable or not.

2.4 Limiting Tensile Strain Method (LTSM)

Widely the most used method to predict settlement damage is the Limiting Tensile Strain Method (LTSM). The method was developed by Burland and Wroth (1974). This empirical analytical damage assessment is based on the maximal tensile capacity of the structural material. The LTSM has been adapted throughout the years. The LTSM as it is being used today is presented in this Paragraph.

The LTSM is basically built up of 6 steps, Burland (1974):

- 1) Greenfield displacements;
- 2) Projection of displacements on the building;
- 3) Determination of the displacement parameters;
- 4) Calculation of the building distortions;
- 5) Combination of building distortions;
- 6) Classification of the building damage;

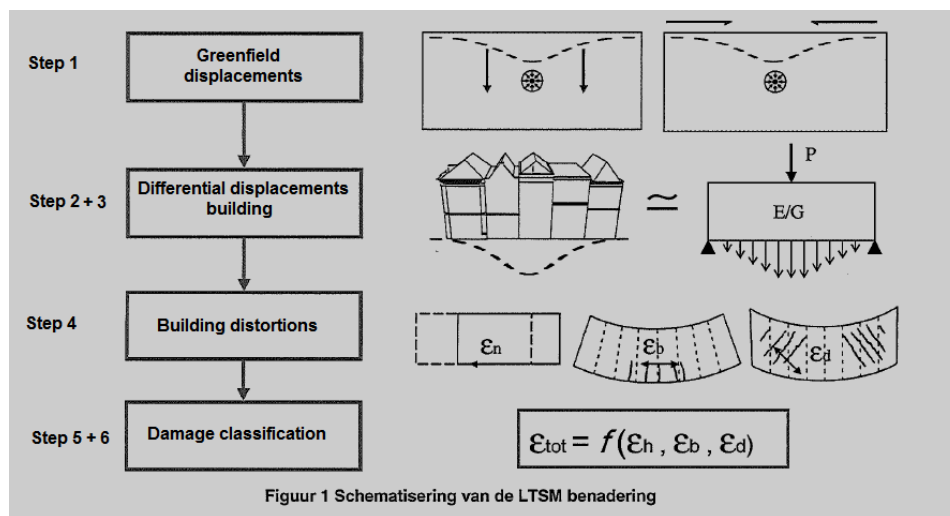


Figure 11: Schematised LTSM, Burland et al. (1974), from Giardina et al. (2010)

Step 1

This step is already discussed in Paragraph 2.2. The settlement profile can be divided into a sagging part (concave curvature of settlement profile) and a hogging part (convex curvature of settlement profile). Theoretically the settlement profile goes to infinity, in practise the 1mm settlement contour is only accounted for since the for part of the building outside this contour the risk of damage is considered minimal, see Figure 12.

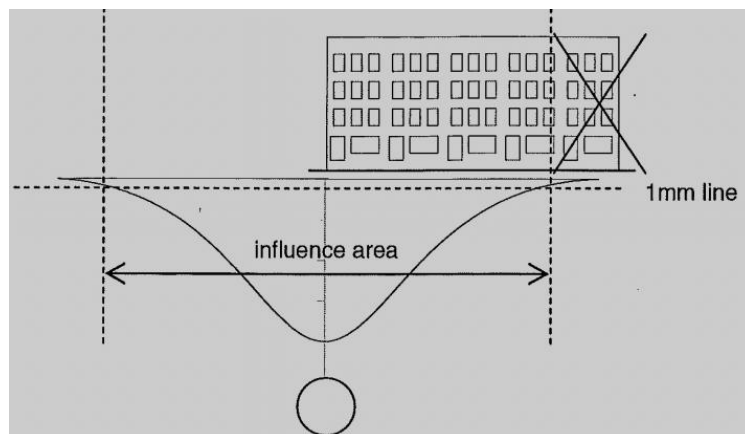


Figure 12: Determination of the influenced part of the building, Mair et al. (1993), from Giardina et al (2010)

Step 2

In this stage the greenfield displacements are known. Now the most common thing to do is to split the building into a sagging part and a hogging part, and determine the L/H ratio of each of these parts. Each part of the building is analysed separately. Netzel (2009) demonstrated the splitting of the building can influence the magnitude of the bending moments and shear forces. It is recommended to split the building in two parts only if the tilt values of each of the two parts do not differ more than 15% with the total tilt. Step 2 is visualized in Figure 13. Netzel (2009) also concluded that cutting of the building at the 1mm contour can lead to an underestimation of the settlement induced damage.

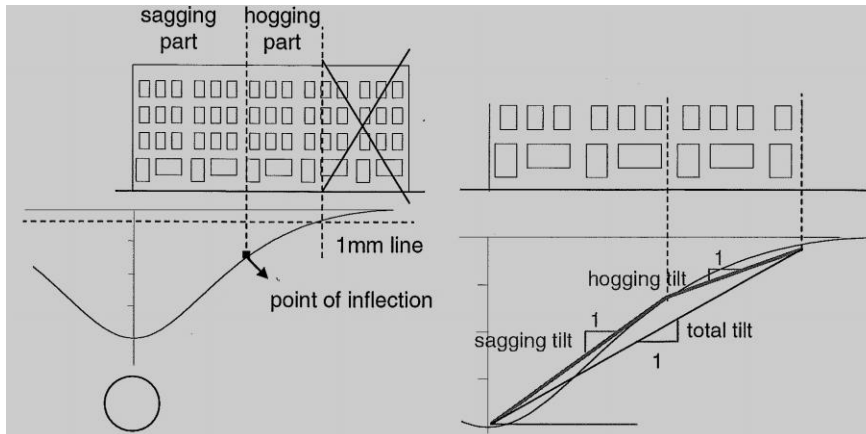


Figure 13: Splitting of the building into a hogging and sagging part, definition of the hogging, sagging and total tilt, Burland et al. (1974), from Giardina et al (2010)

Step 3

The considered building is modelled as a weightless, isotropic, elastic, rectangular beam of length L and height H and a material parameter E/G . H represents the height of the building, measured from the foundation level to the eaves, the roof structure is often ignored. For a massive masonry wall an E/G ratio of 2.6 is often applied (Poisson ratio is 0.3).

The greenfield displacements are applied to the beam, this includes horizontal and vertical displacements of the soil. Now the parameters mentioned in Paragraph 2.3 can be defined for further analyses. Step 3 is visualized in Figure 14.

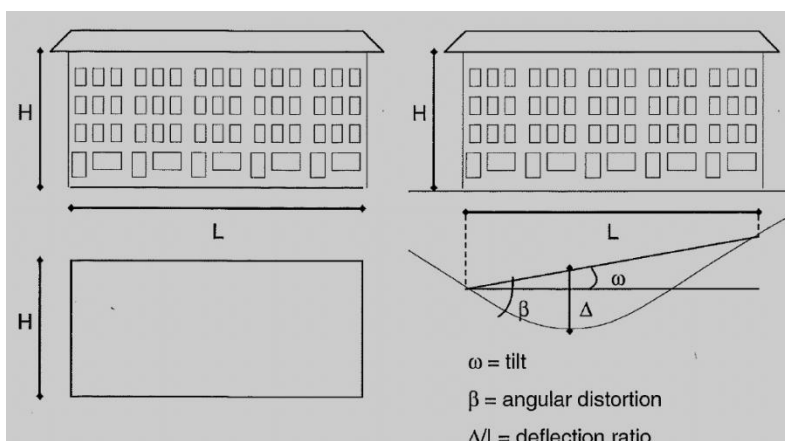


Figure 14: modelling of the building into a beam and the definition of the parameters, Burland et al. (1974), from Giardina et al. (2010)

Step 4

Now the parameters are known, the bending strains and shear (diagonal) strains in the beam can be determined. The different strains are visualized in Figure 15.

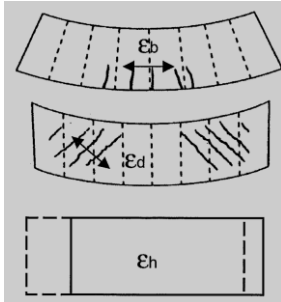


Figure 15: bending, diagonal and horizontal strains, Burland et al. (1974)

The building is modelled as a linear elastic beam on two supports, loaded by a fictitious point load in the middle of the beam, based on the work of Timoshenko (1957).

The horizontal strains are quite easy to determine, the difference in horizontal displacement between two sides of the beam is divided by the total length of the beam:

$$\varepsilon_h = \frac{\delta L}{L} \quad (7)$$

Burland et al. (1974) presented the formulas for the calculation of the bending and shear strains in terms of the deflection ratio (Δ/L), using the formulas introduced by Timoshenko (1957), see Figure 16.

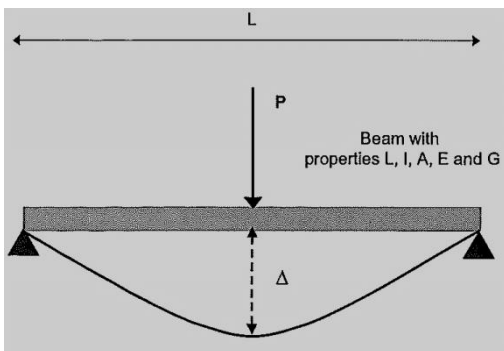


Figure 16: beam model used by Burland et al. (1974)

The deflection of the beam can be calculated by the formula:

$$\Delta = \frac{P * L^3}{48 * EI} + \frac{P * L * \alpha_s}{4 * G * A} \quad (8)$$

In which:

- Δ = deflection of the beam
- L = length of the beam
- G = shear modulus of the beam
- E = Young's modulus of the beam
- I = moment of inertia of the beam
- A = cross section area of the beam
- α_s = form factor

The first term of the equation represents the deflection due to bending; the second term represents the deflection due to shear deformation. The form factor is added to correctly represent the shear and bending contributions. Originally the α_s was defined as 1.5, but according to Netzel (2009) this factor should be 1.2, which would give a better approximation of the shear and bending strains. The maximum bending strains in the beam can be determined with the formula:

$$\varepsilon_{b,max} = \frac{M}{E * W} = \frac{P * L * H}{4 * E * I * 2} \quad (9)$$

In which:

M = bending moment due to point load
 W = section modulus of the beam

Burland et al. (1974) suggested an essential different method to determine the bending strains in the hogging zone and sagging zone: for the hogging zone the neutral axis is placed on the bottom of the beam, for the sagging zone the neutral axis is placed in the middle of the beam. According to Burland et al (1974) the hogging deformation is more susceptible to damage than the sagging deformations, which has been confirmed in empirical studies. Therefore in the hogging mode the neutral axis is placed on the bottom of the beam.

Combining equations 8 and 9 with the right place of the neutral axis gives the following formulas for the maximum bending strains in the beam for respectively sagging and hogging, given in terms of the deflection ratio.

Maximal sagging bending strains:

$$\varepsilon_{b,max} = (\Delta/L) \left[\frac{6 * \left(\frac{L}{H}\right)}{\left(\frac{L}{H}\right)^2 + \alpha_s \frac{E}{G}} \right] \quad (10)$$

Maximum hogging bending strains:

$$\varepsilon_{b,max} = (\Delta/L) \left[\frac{3 * \left(\frac{L}{H}\right)}{\frac{1}{4} * \left(\frac{L}{H}\right)^2 + \alpha_s \frac{E}{G}} \right] \quad (11)$$

A similar approach can be used to define the diagonal strains. The maximum diagonal strains, according to Timoshenko et al (1971):

$$\varepsilon_{d,max} = \frac{1}{2} \gamma_{max} = \frac{1}{2} * \frac{\tau_{max}}{G} = \frac{1}{2} * \frac{3}{2} * \frac{V_{max}}{G * A} = \frac{3 * P}{8 * G * A} \quad (12)$$

In which:

γ_{max} = maximum shear strain
 τ_{max} = maximum shear stress
 V_{max} = maximum shear force

Instead of using the deflection ratio (Δ/L) for the diagonal strains, Netzel (2009) recommended to use the maximum relative rotation (β) for the calculations of the diagonal tensile strains. The relative deflection should be used to determine the tensile bending strains. According to Netzel this

would safer that just using (Δ/L) underestimates the diagonal strains. The deflection ratio and the relative rotation are related through the following equation (Boscardin et al. 1974):

$$\beta = 3 * \frac{\Delta}{L} * \left[\frac{1 + 4 * \frac{E}{G} * \left(\frac{H}{L}\right)^2}{1 + 6 * \left(\frac{E}{G}\right) * \left(\frac{H}{L}\right)^2} \right] \quad (13)$$

Combining the equations gives the following formulas for determining the maximum diagonal strains for the hogging part (neutral axis at the bottom) and sagging part (neutral line in the middle of the beam).

Maximum sagging diagonal strain:

$$\varepsilon_{d,max} = \left(\frac{\beta}{3}\right) \left[\frac{1 + 6 * \left(\frac{E}{G}\right) * \left(\frac{H}{L}\right)^2}{1 + 4 * \left(\frac{E}{G}\right) * \left(\frac{H}{L}\right)^2} \right] * \left[\frac{3 * \left(\frac{E}{G}\right)}{2 * \left(\frac{L}{H}\right)^2 + 2 * 1.2 \frac{E}{G}} \right] \quad (14)$$

Maximum hogging diagonal strain:

$$\varepsilon_{d,max} = \left(\frac{\beta}{3}\right) \left[\frac{1 + 6 * \left(\frac{E}{G}\right) * \left(\frac{H}{L}\right)^2}{1 + 4 * \left(\frac{E}{G}\right) * \left(\frac{H}{L}\right)^2} \right] * \left[\frac{3 * \left(\frac{E}{G}\right)}{\frac{1}{2} * \left(\frac{L}{H}\right)^2 + 2 * 1.2 \frac{E}{G}} \right] \quad (15)$$

Step 5

The governing strain in the building is a combination of the above mentioned horizontal, bending and diagonal strains. The combination of strains can be combined to achieve two values: the combination of maximum bending strain combined with the average horizontal strain ε_{bt} and the maximum diagonal strain combined with the average horizontal strain ε_{dt} . The first one can be determined using simple superposition; the latter on can be determined using Mohr's circle:

$$\varepsilon_{bt} = \varepsilon_{b,max} + \varepsilon_h \quad (16)$$

$$\varepsilon_{dt} = \frac{\varepsilon_h}{2} + \sqrt{\left(\frac{\varepsilon_h}{2}\right)^2 + \varepsilon_{d,max}^2} \quad (17)$$

The highest value of the two combinations is the governing strain within the building. Netzel (2009) advised not to take the horizontal strains into account in the sagging zone. Taking the horizontal strains into account might underestimate the damage due to the imposed settlements.

Step 6

The strains determined in the former step can be used to determine the amount of settlement induced damage done to a building. Boscardin et al. (1989) appointed different damage categories to certain bandwidths of strains, based on the ease of repair. These bandwidths were found by large scale tests of Burland et al. (1974) and other empirical data. The bandwidths and the according damage levels are listed in Table 3.

Category of damage	Damage class	Description of typical damage and ease of repair	Approximate crack width	Limiting tensile strains levels (%)
Esthetical damage	Negligible	Hairline cracks of less than 0.1mm	Up to 0.1mm	0 - 0.05
	Very slight	Fine cracks which can easily be treated during normal decoration. Perhaps isolated slight fracturing in building. Cracks in external brickwork visible on close inspection.	Up to 1mm	0.05 – 0.075
	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repainting may be required. Doors and windows may stick slightly.	Up to 5mm	0.075-0.15
Functional damage, affecting serviceability	Moderate	The cracks require some opening up and can be patched by a mason. Recurred cracks can be masked by suitable linings. Repainting of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weather tightness often impaired.	5-15mm	0.15-0.3
	Severe	Extensive repair work involving breaking out and replacing sections of walls, especially over doors and windows. Windows and doorframes distorted, floors sloping noticeably. Walls leaning or bulging noticeably. Windows broken with distortion. Danger of instability	15 to 25mm	>0.3
Structural damage	Very severe	This requires a mayor repair involving partial or complete rebuilding. Beams loose bearing. Danger of instability.	Usually >25mm, but depends on number of cracks	

Table 3: Bandwidths for limiting tensile strains related to damage (Bre (1981), (1990) and Burland et al. (1989))

Initial damage

It is possible that buildings already have suffered from loads acting on the structure, due to for example bad construction, rotten (timber) piles, long term settlements, earthquakes, etc. An already damaged building could be more sensitive to settlements than a recently constructed (non-damaged) building. Therefore the SBR (1998) introduced empirical bandwidths to take into account the initial condition of the building. The table below shows in which cases the limit tensile strains have to be reduced. Though there can be a wide variety of damage patterns, this table can only be used as an indication to the increase damage susceptibility.

Building condition	Reduction of tolerable strain limits
Poor building condition versus good building condition	55-75%
Moderate building condition versus good building condition	20-30%

Table 4: Reduction of the limiting strain due to initial state of the building

Limitations of the LTSM

The LTSM is based on major assumptions and simplifications. It is said the LTSM leads to conservative damage predictions while Netzel (2009) proved this is not always the case. The most important assumptions and simplifications are listed below:

- A completely decoupled method is chosen, no soil-structure interaction is taken into account.
- The building is represented as a linear elastic beam on two supports. Non-linear behaviour such as stress and stiffness redistribution is not taken into account. Brittle behaviour and localized cracking are not taken into account.
- Façade like structures can only be represented in a different E/G ratio, which is not representative for a façade. Stress peaks at the corners of doors and windows are not taken into account, which are weak spots in façade structures.
- It does not include other structures than masonry like concrete structures, although Netzel (2009) proposed an alternative for damage assessment on frame-like structures.
- It does not include 3D effects, such as floors or adjacent (internal) walls, which create a local higher stiffness.
- Other loads acting on the building are not taken into account (live/dead loads).
- Initial damage can only roughly be taken into account by the reduction of tolerable strain limits, while different damages can influence the behaviour of buildings in different ways.
- The settlement due to the (time dependent) longitudinal settlement trough is not taken into account. The longitudinal trough could introduce damage as well.

2.5 Numerical models

The modern computers and computer technology provide a powerful tool to make a more detailed damage assessment; numerical modelling. Using the Finite Element Method (FEM), a building, the soil and excavations can be brought together in one numerical model, providing the chance to take into account for instance the soil structure interaction. Modern powerful computers can handle bigger and more sophisticated models than ever before, a lot of research is being performed on 2D and 3D models of buildings subjected to imposed settlement. The results prove that numerical models can give more reliable results than the above mentioned LTSM, since a lot of simplifications/assumptions in for instance the LTSM can be eliminated. Although the results are reliable, numerical models are not always used yet because of the required computational and modelling efforts.

A short overview of the modelling techniques is presented in this Paragraph.

Greenfield analyses

The tunnel induced settlements can be modelled in a numerical model. A soil model is adopted and within the soil a tunnel is modelled. Modelling the soil can be done in several ways:

- Linear elastic isotropic soil conditions.
- Linear elastic soil with increasing Young's modulus at increasing depth.
- Non-linear elastic plastic soil
- Multi surface plasticity soil.
- Spring model

Also the tunnel can be modelled in different ways:

- Remove soil elements and apply radial stresses on the tunnel boundary.
- Remove soil elements and lining activation.
- Remove soil elements, lining activation and application of radial stresses on the boundary.
- Contraction of the tunnel area.

All combinations of the tunnel and soil model are possible. It is known that numerical modelling techniques usually give a wider settlement profile than the Peck-formula describes, which could affect the results.

The different models will not be further elaborated in this thesis since they will not be used.

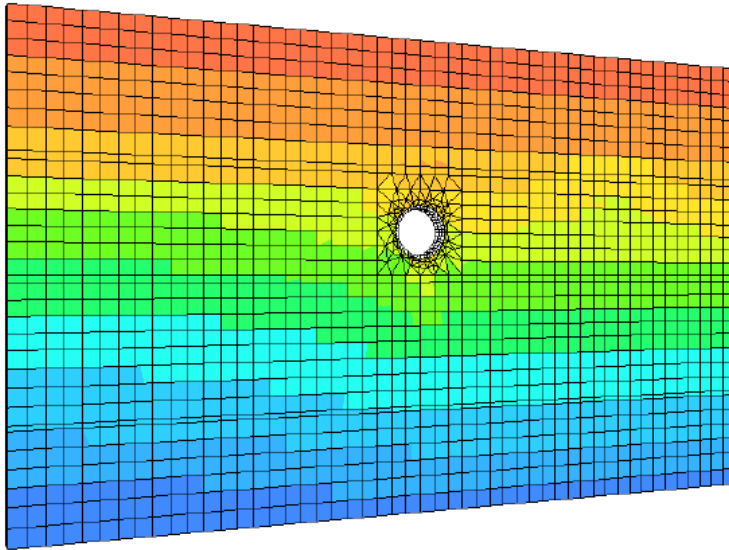


Figure 17: example of a 2D greenfield settlement prediction model, used for the SRA of the North/Southline.

Uncoupled model.

In this model the building and the soil are fully decoupled. The displacements obtained by the greenfield analyses are directly applied on the building. The building is thus assumed to directly follow the greenfield displacements. The building can be modelled in several ways.

First of all the, as in the LTSM, the building can be modelled as a beam with a length, height and an E/G ratio. This E/G ratio can be adapted to simulate openings in for instance a façade, like doors or windows. As in the LTSM, this model does not correctly represent a façade but it could be useful for a solid (masonry) wall.

The second option is to represent the building as a façade, so the building model has to include windows and doors. This will give a more realistic representation of the building, the corners of doors and windows now can cause for instance stress hotspots at the corners of these elements. Also stiffer parts of the building, like lintels, can be taken into account. Including structural details like transversal walls and openings allows predicting of the crack pattern, which is strongly affected by the discontinuities.

The third variation can be obtained by the modelling of the masonry itself. The masonry can be modelled as a linear elastic material, which neglects nonlinear effects. This model can vary a lot from the real damage obtained by field data, since no stress or stiffness redistributions are taken into account. The model of the masonry can be improved by modelling the masonry as a non-linear material, which is more realistic but more difficult to model. Netzel (2009) recommends using (non-linear) smeared crack models since they have a significant influence on the post-cracking behaviour of a structure. A numerical prediction with elastic strains leads to an underestimation of the damage due to imposed settlements. Within the non-linear model a variation of parameters is possible: different crack models and different softening models are available. In practice often a smeared cracking model in combination with a linear softening crack models is used.

The fourth variation is the mesh-size of the model. Big element for instance could give other damage results than small elements, and either under- or overestimation of the damage. According to Netzel (2009), the difference in damage between a rough and fine mesh is rather small in a 2D model. A great downside of the uncoupled model is the absence of soil-structure interaction, which could influence the settlement profile.

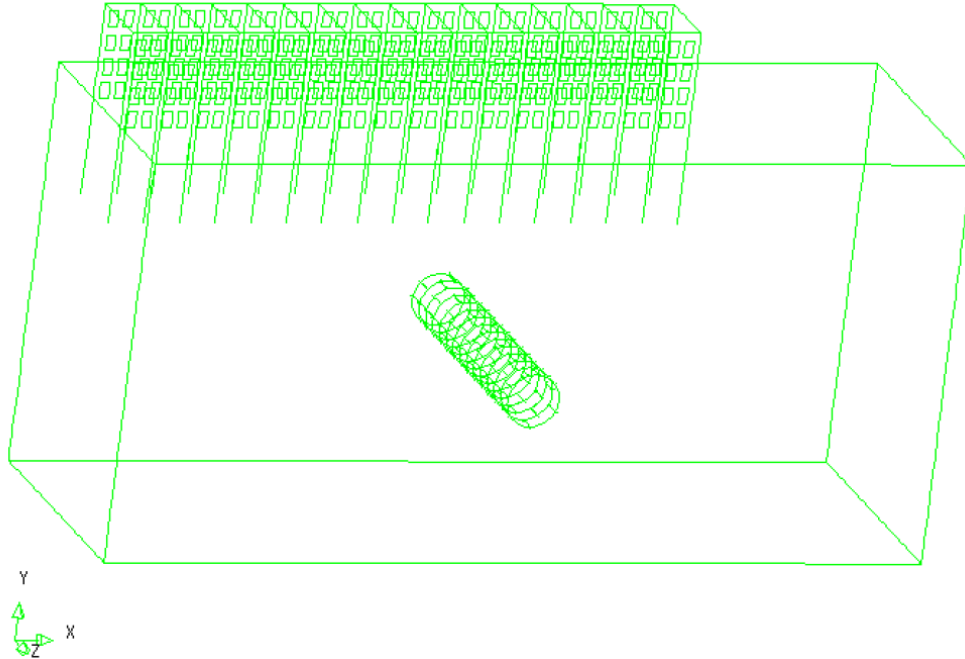


Figure 18: example of a fully couple 3D model with foundation geometry, created by Witteveen+Bos.

Fully coupled model.

In a coupled model the soil, tunnel and the structure are combined in one model. Interface elements can be applied to model the soil structure interaction, or the whole geometry of the foundation has to be modelled. The above mentioned variations in soil, tunnels and building modelling techniques can be applied; 2D and 3D coupled models are possible.

2D model.

In a 2D model the foundation can be represented by interface elements with a normal and tangential stiffness. A distinction can be made between 'smooth' and 'rough' interface elements. The smooth interface basically assumes no friction between the soil and the building; horizontal ground movements do not affect the building. According to Netzel (2009) a smooth interface should be used in the sagging zone for a conservative damage prediction, because horizontal ground movement gives a negative moment in the sagging zone, leading to less damage. In a rough interface the friction between building and soil is modelled, resulting horizontal ground movements are applied to the building (in a 2D model). This rough interface is more realistic than the smooth interface. For a conservative approach the rough interface should be applied in the hogging zone only, since horizontal ground movements introduce horizontal tensile strain, increasing the amount of damage (Netzel, 2009). One can choose to model the whole foundation geometry as well, but this is more time consuming.

A disadvantage of the 2D model is that it cannot represent 3D action of the modelled house, the soil or the advancing tunnel. For a more accurate result, a 3D model should be created.

3D model

For a 3D model a rough interface is advisable, but a smooth interface can be applied as well. Modelling the soil structure interaction via interface elements gives the opportunity to modify the

foundation parameters without actually modelling the foundation geometry. The shear behaviour of the interface, which models the transmission of the horizontal deformation of the soil onto the building, was proven to affect the damage mechanism significantly.

Instead of using interface elements, one can decide to incorporate the whole foundation geometry of the foundation.

The coupled model is more difficult to build but it can represent the soil-structure interaction and therefore could give more reliable results. It will cost a lot of computational effort.

Semi coupled model

In a semi coupled model only the building is represented. Instead of applying the greenfield displacements directly on the building, the greenfield displacement are applied to interface elements connected to the structure. This model is an intermediated form of the former two. It represents some interaction between soil and structure and is easier to model than the fully coupled model. The semi coupled model seems a better solution to represent the soil structure interaction, but is sensitive to interface parameters (Giardina et al. (2008)). This model will be used in this thesis.

Application of dead and live loads

The self-weight of the building and the live loads acting on the building pose a load on the foundation of the building and consequently on the soil it's founded. These loads can cause an initial stress state which can influence the settlement profile and accordingly the amount of damage. The loading conditions can be applied to the building in the numerical model. In the first time step the dead load is imposed on the soil, causing an initial stress state. After this first step, the tunnel is modelled in the soil. Netzel (2009) stated the introduced stresses in a massive masonry wall are relatively small, but the introduced stresses are depending on the soil stiffness.

Another live load is the advancing TBM. The advancing tunnel causes a longitudinal through and this longitudinal through could cause (permanent) damage to a building. The advancing tunnel can only be modelled in a 3D model. Adopting a smeared crack model tension softening model for the masonry permits to reproduce the relation between the tunnelling advance and the volume loss increase, and the crack initiation and propagation. This gives a reliable crack pattern and a good indication of the crack width order of magnitude in 3D models.

Imposed settlement over the years due to for instance soft soil conditions or poor foundation conditions, could introduce cracks or other forms of damage into the building. This kind of initial conditions can be implied in the numerical model, for instance an initial stress state could be introduced at certain point within the structure. This sure leads to other crack initialisation than for an un-cracked building.

Soil structure interaction

Soil structure can play an important role in damage assessment. The building imposes stresses on the soil, and the geometry of the building can interfere with the tunnel induced settlements. The soil structure interaction can affect the longitudinal and transversal settlement trough. The amount of soil structure interaction is highly dependent on several parameters like the building geometry and the type and stiffness of the soil.

Coupling building and the bored tunnel in one model makes it possible to represent the soil structure interaction. Not only the effect of settlement on the building, but also the effect of the building on the settlement profile is captured (3D).

3. Case study Daniel Stalpertstraat.

An important part of this thesis is the case study. The Daniel Stalpertstraat in the district “De Pijp” is studied in detail. Not often a TBM passes right underneath a row of houses; in most cases this is avoided to reduce the risk of damage. Moreover the building and soil movements are extensively monitored, providing a unique opportunity to study the building response to tunnel induced settlements. The case study can later on be used for the numerical model in Chapter 4.

In the case of the Daniel Stalpertstraat the tunnel alignment runs perpendicular to the alignment of the facades of the houses. This provides the situation of a transversal settlement trough under a row of facades. As was discussed in Chapter 2, one of the shortcomings of the LTSM is that it does not take into account the openings in the façade (weak spots) with according reduced shear stiffness. The results of this case study can be useful to improve for instance the LTSM for the case of facades.

The case study is built up of 4 parts. First a short description of the house and the tunnelling process in the Daniel Stalpertstraat is given. In the second part the monitored data of soil settlements and horizontal and vertical movements of the buildings in Daniel Stalpertstraat are analysed. Also the behaviour of the building movements over time is treated. In the third part briefly the results of the measurements of two adjacent streets are compared to the results of the Daniel Stalpertstraat. In the fourth part the field data is used to find increased settlement troughs for further analyses. After that the conclusions and recommendations of the case study are summarized.

3.1 Description Daniel Stalpertstraat

The houses in the Daniel Stalpertstraat have been built at the beginning of the 20th century. They are traditional small but high Amsterdam houses; a picture of the houses can be found in Figure 19. The foundation of the houses, as in most Amsterdam houses, exists of wooden piles that are based on the first sand layer, which is roughly situated at 12 m below the soil surface. The western tunnel tube of the North/Southline passes right underneath these houses, as can be seen in Figure 20, and has a diameter of 7.08 m.

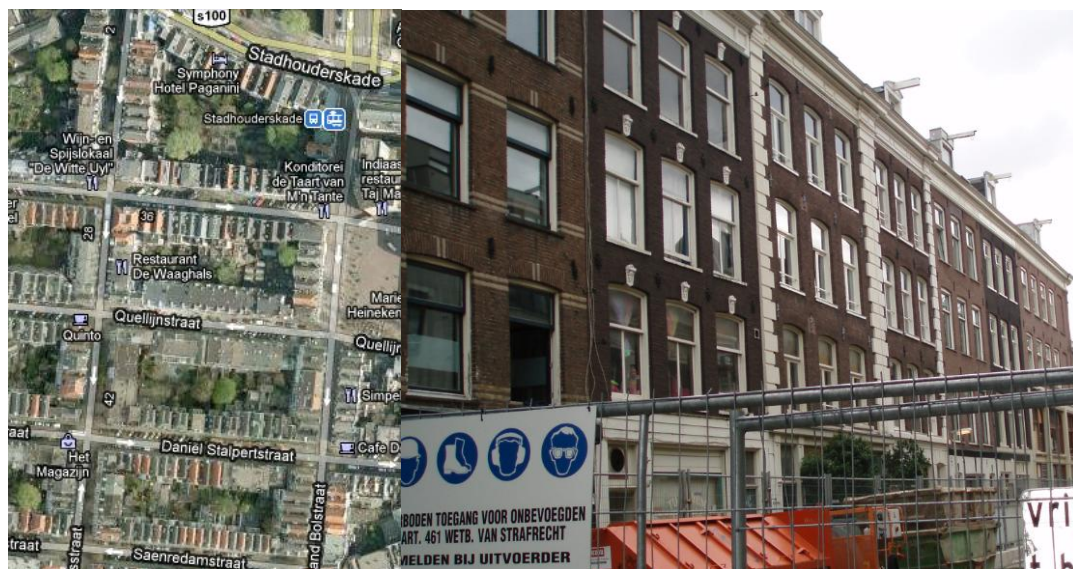


Figure 19: Satellite image (Google Earth) and a photo of the Daniel Stalpertstraat

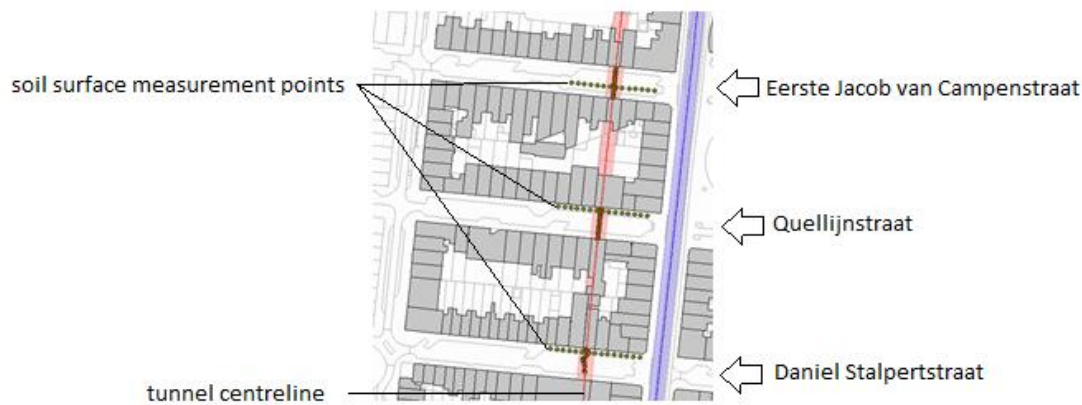


Figure 20: Alignment of the tunnel (eastern tunnel tube blue and western tunnel red) and the measurement points of the soils surface settlements in the Eerste Jacob van Campenstraat (u), Quellijnstraat (m) and Daniel Stalpertstraat (d)

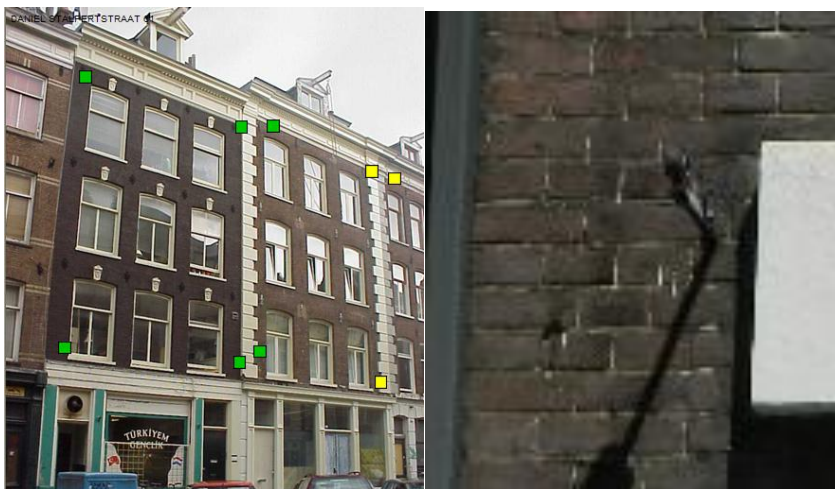


Figure 21: Measurement prisms attached to the building (l) and a picture of one prism (r)

3.2 Analyses measurements Daniel Stalpertstraat

The soil surface movements are monitored before, during and after the TBM passage. Using the vertical movements of measurement points on the street, the transversal soil settlement trough at soil surface due to the tunnelling process can be determined. Horizontal and vertical building movements are measured with 4 measurement prisms positioned on the façade of each house, placed in a rectangular pattern. This is only done for the houses in the area affected by the tunnel. The measurements at 1, 2, 3, 4 and 8 weeks after passage of the TBM will be used for the analyses. After week 3 a deep pit situated in the Daniel Stalpertstraat was filled with sand and caused additional settlements, because of this the settlements after 3 weeks are not solely caused by tunnel induced settlement but a combination of both cases. The methods for extraction of the results are elaborated in Appendix 1.

3.2.1 Vertical soil and building movements

The vertical soil surface settlements are extracted from the database. Unfortunately the horizontal soil displacements were not measured. The transversal soil surface settlement trough is found by applying a Gauss curve to the measured vertical displacements of the soil surface.

The measurements of the horizontal and vertical building movements are collected and analysed. The building response is fitted with a Gauss curve as well to reduce the influence of measurements

errors. The most suitable Gauss curve (Gauss fit) is found using the method of the least squares. The Gauss curve is described by the formula by Peck (1969) in Chapter 2.2.

Because the settlements of the first sand layer are not measured directly, these have to be calculated. This can be done by using Peck's formula, but Witteveen+Bos ran in the SRA of the North/Southline several numerical analyses to determine the settlements at soil surface, first and second sand layer at different volume losses. In these numerical models the exact built up of the soil was taken into account as well as the dead weight of the buildings; therefore the results can be considered to be reliable. The results of these analyses combined with the measurements of the soil surface settlements are used to determine the transversal settlement trough of the first sand layer. The calculation can be found in Appendix 2.

The results of the measurement are collected for week 1, 2 and 3 after passing of the TBM. After that an additional settlement is introduced by the filling of the pit and the measurements are no longer the result of the tunnelling process only. This will be treated later. The results of week 1 and 3 are given in Figure 22 and Figure 23. The according Gauss curves are given in the graphs as well. Results of week 2 can be found in appendix 1.

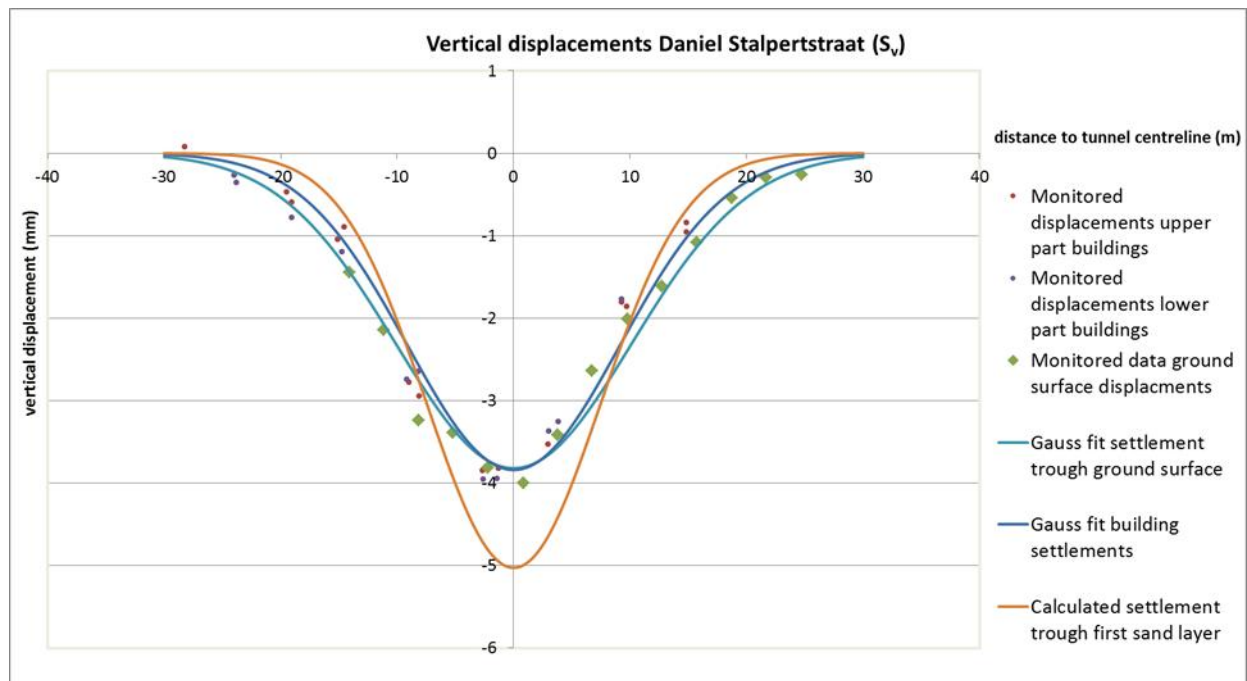


Figure 22: Vertical movements of the soil and buildings of the Daniel Stalpertstraat after 1 week.

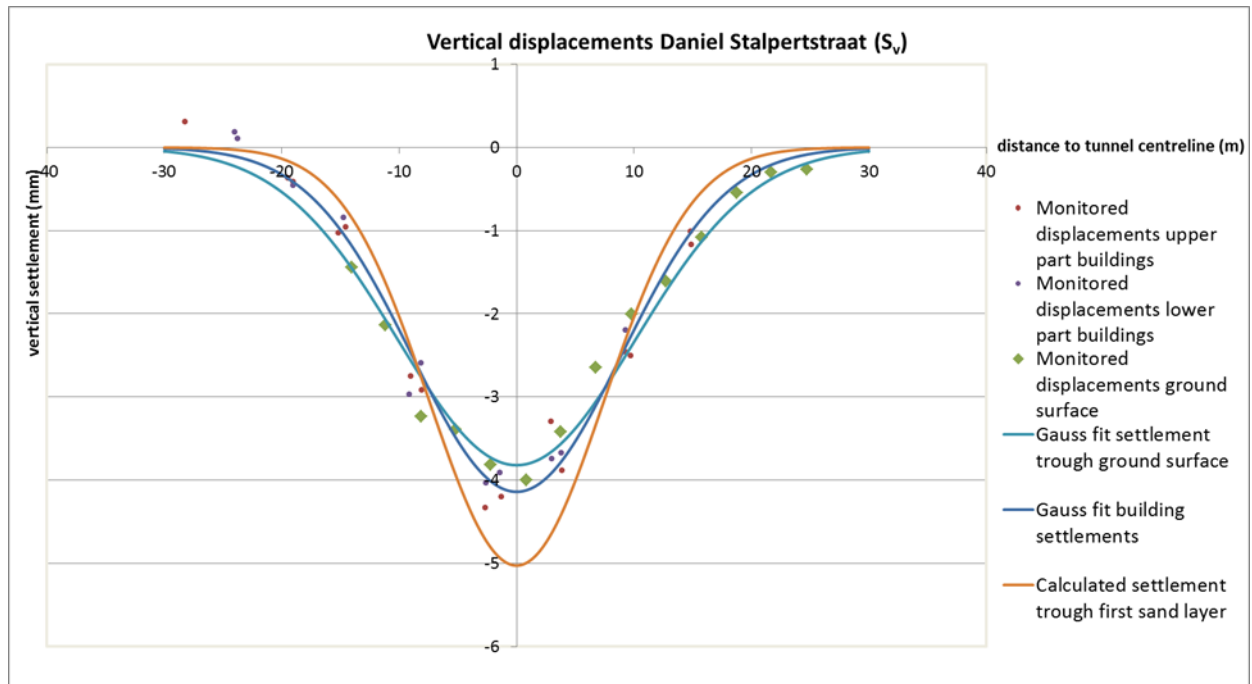


Figure 23: Vertical movements of the soil and buildings of the Daniel Stalpertstraat after 3 weeks

The figures show that the applied Gauss fits do not differ much from the measured data. The characteristics of the Gauss curves over the first 3 weeks are given in Table 5. As can be obtained from the table, the vertical building movements increase over time whereas the soil settlements do not increase anymore. The vertical building displacement profile becomes steeper over time. The maximum vertical settlement of the facades lies between the maximum settlement of the ground surface and first sand layer.

	Week 1		Week 2		Week 3	
Gauss curve	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)
Soil Surface	-3.76	9.98	-3.84	10.13	-3.82	10.09
First sand layer	-5.03	7.43	-5.03	7.43	-5.03	7.43
Vertical displacement building	-3.84	9.12	-3.77	8.70	-4.14	8.82

Table 5: Characteristics of Gauss curve of the soil and building settlements up to 3 weeks after passing of the TBM

After week 3 a deep pit, used for mitigation measures, was filled with sand. This introduced extra settlements of the soil and extra building movements. The pit is located 5-10 meters to the right of the tunnel centreline; a satellite image of the pit is provided in Figure 24. Although the measured movements are no longer the result on the tunnelling process only, the data can provide insight on the movements of the building over time.



Figure 24: Pit for mitigation measures and the tunnel centreline (red) in Daniel Stalpertstraat, Google Earth

The data at 1 week after filling of the pit (4 weeks after passing of the TBM) and 4 weeks after filling of the pit (8 weeks after passing of the TBM) have been collected as well. The measurements and according Gauss curves after week 8 have been provided in Figure 25. The graph of week 4 can be found in appendix 1.

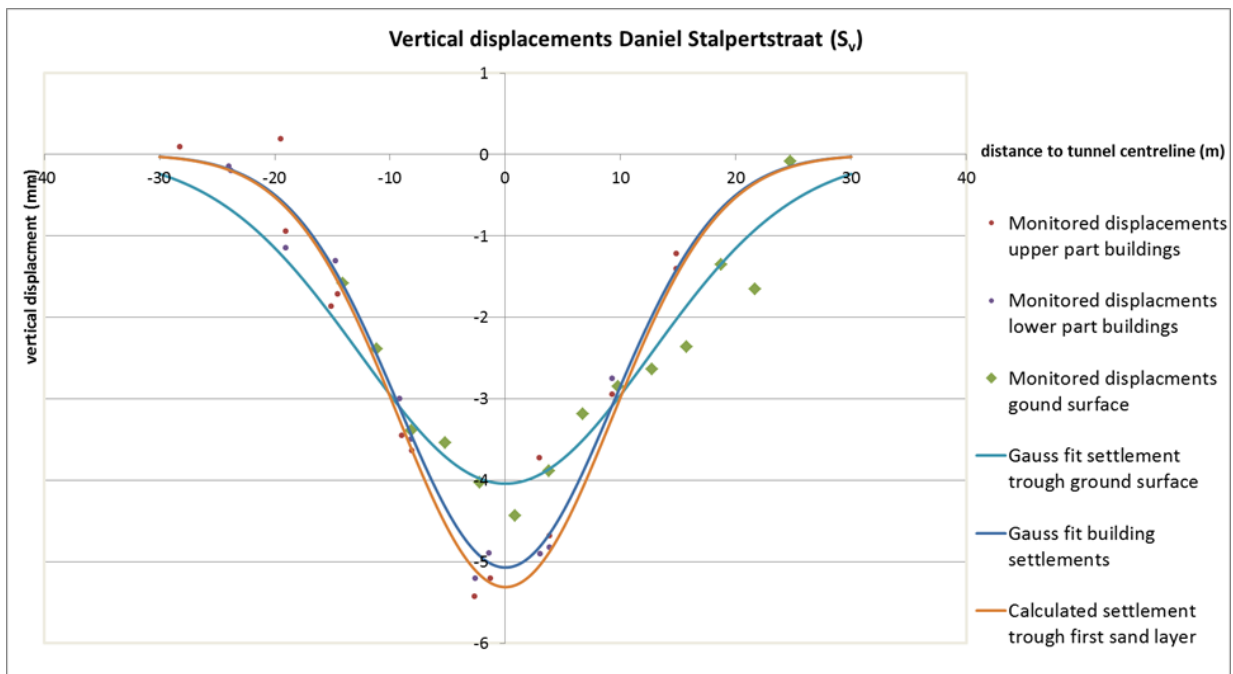


Figure 25: Vertical movements of the soil and buildings of the Daniel Stalpertstraat after 8 weeks

The soil settlements at the right side of the tunnel centreline increased more than the left side, which is caused by the position of the filled pit. Therefore the applied Gauss curve for the soil surface settlements does not fit well anymore. The displacements are a sum of 2 troughs; one of the tunnel induced settlements and one of the pit induced settlements. The results of the Gauss curves are given in Table 6. These measurements can't be used for further analyses of tunnel induced settlements; it does show that the vertical building movements increase over time.

	Week 4		Week 8	
Gauss curve	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)
Soil Surface	-4.04	12.53	-4.11	12.61
First sand layer	-5.31	9.29	-5.31	9.29
Vertical displacement building	-4.60	9.11	-5.07	9.28

Table 6: Characteristics of Gauss curve of the soil and building settlements 4 and 8 weeks after passing of the TBM

The movements of the houses in the Daniel Stalpertstraat increase over time while the soil surface settlements remain constant. The vertical settlements of 3 points of the facades give a better insight on the increase of vertical movements of the building over time (Figure 26). The increase of vertical building movements indicates that the bearing capacity of the piles of these houses is partly based on adhesion of the soil; the bearing capacity has nearly reached its maximum. This is not a direct result of the tunnelling; the bearing capacity was in this state before the tunnelling process started. The applied Gauss curves for the soil and building settlements indicate that the maximum vertical settlement of the building is in between the maximum settlement of the soil surface and the first sand layer. This was measured 3 weeks after passing of the TBM, just before the settlement increase due to filling of the mitigation measures pit. The filling of the pit introduced extra settlements. It is uncertain whether the buildings stopped moving due after week 3 due to tunnel induced settlements only.

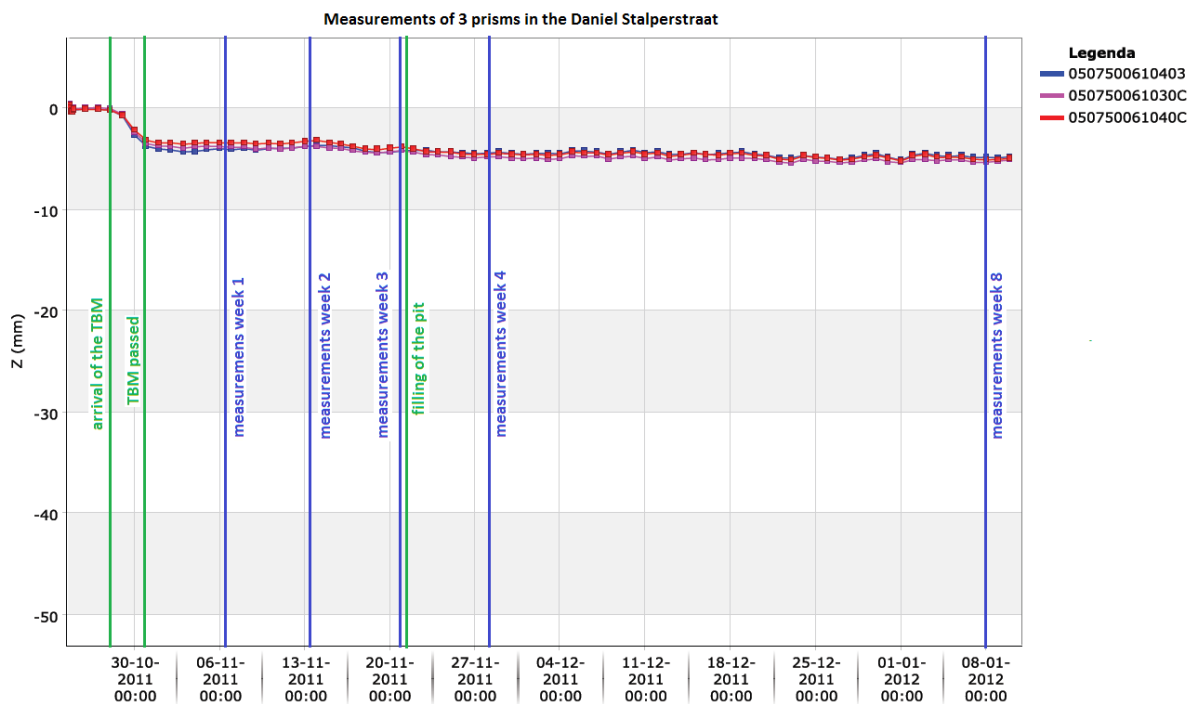


Figure 26: Mile stones in the Daniels Stalpertstraat

3.2.3 Horizontal building movements

The horizontal movements of the houses in the Daniel Stalpertstraat are very small, within the margin of error of the monitoring system (1 mm). The differing horizontal displacements over time do not show a pattern, like the vertical displacements do; the difference is mainly caused by the less accurate measurement of the horizontal building movements. The exact value of the horizontal building displacements therefore is less accurate. This is further elaborated in Appendix 1.

The measured horizontal displacements of week 3 are given in Figure 27. From the Gauss curves of the vertical soil settlements of the first and second sand layer a theoretical horizontal displacement profile is determined through the relation by Peck (1969), described in Chapter 2.2.

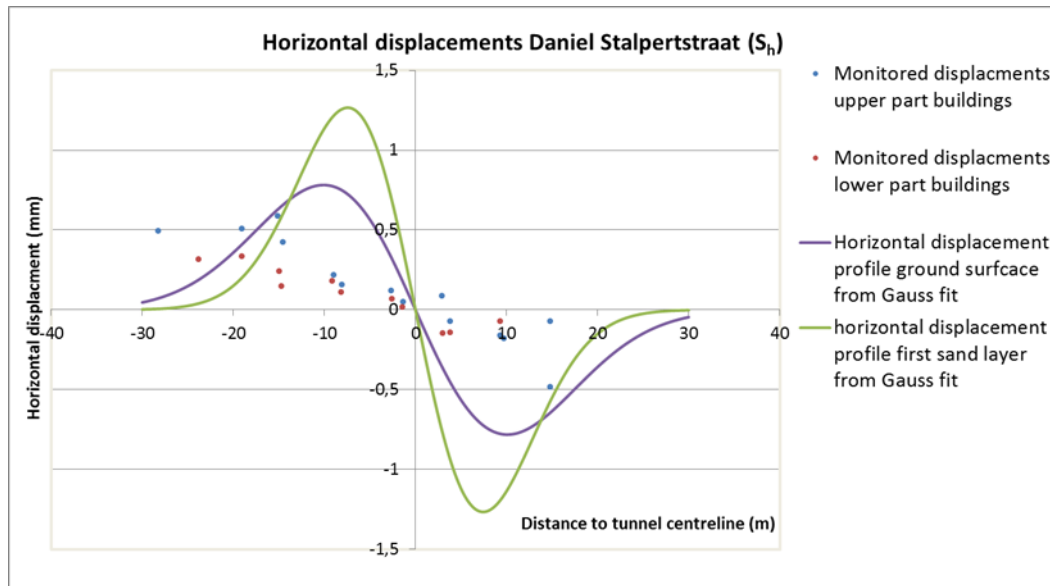


Figure 27: Horizontal movements of the buildings and the horizontal displacement profiles after 3 weeks

As said before, the values of the horizontal building movements are less accurate. The trend of the horizontal displacement does not show good comparison with the profile deduced from the vertical settlement trough.

3.2.4 Damage evaluation.

Before the passing of the TBM the initial cracks in the facades of the Daniel Stalpertstraat have been measured (length and width) and documented in drawings. After passing of the TBM the cracks were examined again. The cracks did not open or increase in length, nor did new cracks occur. The tunnelling process did not introduce damage to the facades.

3.3 Comparison of results with other streets

For the streets adjacent to the Daniel Stalpertstraat, the Quellijnstraat and the Eerste Jacob van Campenstraat, the data has been collected as well. The houses situated in these streets are similar to the ones in the Daniel Stalpertstraat. The foundation level differs in the case of the Eerste Jacob van Campenstraat. A full elaboration of the comparison can be found in Appendix 1. The layout of the streets can be found in Figure 20.

The measurements of the movements of the houses and soil surface of these streets are collected for comparison; it is useful to check whether the building in other streets behave in a similar way as the buildings in the Daniel Stalpertstraat. The data is collected 1, 4 and 8 weeks after passing of the TBM. The results are given in Table 7 and Table 8.

<i>Quellijnstraat</i>	Week 1		Week 4		Week 8	
Gauss curve	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)
Soil Surface	-2.48	8.49	-2.48	8.49	-2.48	8.49
First sand layer	-3.5	6.32	-3.5	6.32	-3.5	6.32
Vertical displacement buildings at the north side	-2.46	8.11	-2.71	8.46	-2.49	7.68
Vertical displacement buildings at the south side	-2.39	10.81	-2.76	10.76	-2.61	10.09

Table 7: Results of the Gauss curves for the Quellijnstraat

<i>Eerste Jacob van Campenstraat</i>	Week 1		Week 4		Week 8	
Gauss curve	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)	$S_{v,max}$ (mm)	i_x (m)
Soil Surface	-3.82	10.09	-3.82	10.09	-3.82	10.09
First sand layer	-5.03	7.43	-5.03	7.43	-5.03	7.43
Vertical displacement buildings at the north side	-2.99	10.87	-2.79	10.08	-2.93	10.09
Vertical displacement buildings at the south side	-2.81	13.11	-2.82	9.44	-2.82	9.44

Table 8: Results of the Gauss curves for the Quellijnstraat

For a better understanding of the differences of the measurements with the ones of the Daniel Stalpertstraat, the graphs of the measurements of the Quellijnstraat and Eerste Jacob van Campenstraat can be found in Appendix 1.

The tables show that the depth and width of the transversal soil settlement trough differs per street. This could be the result of a different soil built up, TBM performance or soil structure interaction.

Secondly the buildings response is different in every considered row of facades. The conclusions that can be drawn from the results are listed:

- From the measurements it appears that the building response due to tunnel induced settlements is influenced by the quality of the foundation. It seems that the buildings respond stiffer in case of a better foundation quality, which could also have an influence on the soil-structure interaction. Usually in case of a good quality foundation the hull is not weakened by cracks either. This could result in stiffer building response. It should however not be neglected that the geometry of the facades as well as the properties of the masonry could have an influence on the building response as well. The building response depends on many factors, not one factor can be said to be decisive for the building response;
- Where the houses in the Daniel Stalpertstraat show increasing building movements over time, the houses in the other streets don't. The bearing capacity of the piles in of the houses in the Daniel Stalpertstraat seems to be less than all the other ones.

- The foundation level of the houses has a significant influence on the building response to tunnel induced settlements;
- The horizontal displacements are less accurate and do not show good comparison with the applied horizontal displacement profile;

The findings of the Daniel Stalpertsstraat cannot be confirmed by looking at the measurements from other row of houses in “De Pijp”. The building all show different behaviour than the houses in the Daniel Stalpertsstraat.

A full elaboration of the results of the adjacent streets can be found in Appendix 1. Because the results of the adjacent streets will not be used in further analyses, this part is not treated extensively in the report.

3.4 Conclusions and recommendations case study

The conclusions that can be drawn from the analyses of the monitored data are listed:

- The settlements of the facades in the Daniel Stalpertstraat increase over time, not only due to the filling of the pit. This is probably caused by the bearing capacity of the piles which have almost reached their maximum. The maximum vertical building movement lies in between the maximum vertical settlement of the soil surface and first sand layer.
- The applied Gauss curve matches the soil surface settlements; the Gauss curve is a good way to determine the transversal settlement trough characteristics.
- The measured horizontal displacements are less accurate because the values are within the margin of error of the measurement system. The exact values will not be used in further analyses.
- Filling of the pit with sand introduces extra soil and building settlements; an additional 0.2 to 1mm is reached in 4 weeks' time, which is an increase of 25%.
- The building response differs per row of houses. Not one cause only can be appointed for this difference in building response, although buildings with a decent foundation quality seem to react stiffer than those with a foundation of less quality.
- The buildings in streets adjacent to the Daniel Stalpertstraat do not show an increase of movements over time.
- Foundation at different depths within a house block can give rise to problems because of the difference in settlement between the houses. This difference can cause additional damage to the facades.
- The settlement did not cause damage to the facades of the houses.

The recommendations following from the analyses:

- Keep monitoring the building displacements after passing of the TBM, building settlements could increase due to time dependency of the building settlements. A long close out of the monitoring is advised, for instance half a year, to ensure the building has come to a standstill.
- The settlements of the first and second sand layer should be monitored to obtain more knowledge of the development of soil settlement over the height of the soil. This could verify numerical calculations of soil settlements.

3.5 Assumptions for further analyses

Since the data of the other streets cannot prove whether the houses in the Daniel Stalpertstraat stopped moving after week 3, it is assumed that this is the case. Since the houses in other blocks span the transversal settlement trough at first sand layer, it would be logical that the houses in the Daniel Stalpertstraat are able to span the soil settlements at first sand layer as well. The measurements of the vertical settlements in week 3 are assumed to be reliable and will be used for further analyses.

The horizontal soil displacement profile determined from the transversal settlement trough is used for further analyses as well.

Since the predicted settlements of the first sand layer and soil surface in the Daniel Stalpertstraat were not the same as obtained by measurements, a calculation is made for the settlement profile of the first sand layer. Although the profile seems correct it is not sure that the profile is consistent with reality. For further analyses the calculated settlement profile at first sand layer is assumed to be reliable.

4. Numerical 2D Model

The second step of the thesis is the verification of the measurement found in the case study, using a 2D numerical model with the geometry of the facades in the Daniel Stalpertstraat. In this chapter the numerical 2D model will be given the properties to match the measured movements of the facades. The finite element (FE) program DIANA is used to create a numerical model of the facades of the Daniel Stalpertstraat. The geometry of the facades is measured at site and applied to the numerical model; this should provide a reliable model. A semi-coupled analysis will be used for the model, with interface elements representing the foundation of the houses. The numerical model should result in applicable model properties. This calibrated 2D numerical model will be used for further analyses. As was concluded in the case study, the imposed settlements did not cause the facades of the Daniel Stalpertstraat to crack. Using the field data and numerical analyses of Witteveen+Bos an extrapolation of the measured soil settlements is made. This results in larger settlement troughs that can be used for further analyses. In the last paragraph the conclusions from the numerical model will be summarized.

4.1 Numerical model description

In this paragraph the 2D numerical plane stress model that simulates the facades in the Daniel Stalpertstraat is described.

Geometry and materials

As mentioned in the introduction, the geometry of facades in the Daniel Stalpertstraat was measured at site in order to obtain an accurate numerical model. The outline of the numerical model is given in Figure 28. A semi-coupled analysis is used to simulate the behaviour of the facades in the numerical model due to tunnel induced settlements. The average height of the facades is 14 m; the total length of the model is 65m.

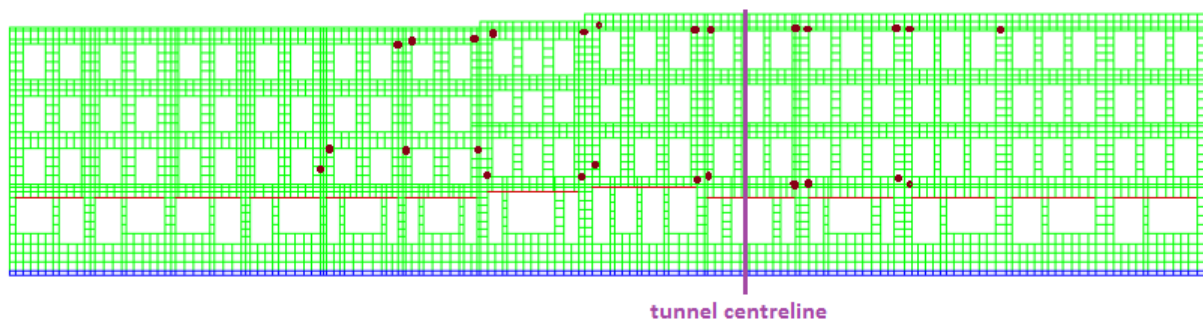


Figure 28: Numerical model of the facades, with; masonry (green), lintels (red) and interface elements (blue) with the measurement prisms (brown)

The model is built up of three parts: foundation, masonry and lintels. The foundation is represented by interface elements attached to the bottom part of the building. The settlement trough at first sand layer will be applied to the facades through these interface elements. The masonry of the building is built up of three parts; a foundation layer, masonry at ground floor level and masonry further to the top of the buildings. Each layer has a different stiffness; the assumed thicknesses are often found in a traditional Amsterdam house. The thickness of each of the layers is given in Figure 29. For the interface elements unit width is assumed.

The assumption of the width of the interface has an influence on the relative stiffness between the facades and the soil. If the width of the interface elements is increased, the participating width of the soil is increased. In order to obtain the same results for a larger participating width, the stiffness of the façade has to be increased as well. The relative stiffness of the walls and interface has to remain

the same. The influence of the width of the interface is not further studied in this thesis, but it should be kept in mind while reading the report.

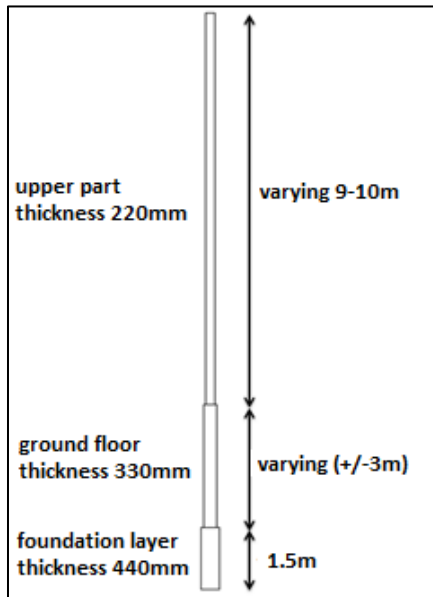


Figure 29: Thickness of different parts of the facades

The foundation layer is estimated to have a height of 1.5m, which is common for typical Amsterdam houses. The lintels are applied at the places where they are positioned in the Daniel Stalpertstraat; the lintels used are steel beams.

The elements used in the numerical model are given in Table 9. The size of the elements is approximately 0.40m. A 2x2 integration scheme is used.

Model part	Elements type	Element name
Masonry walls	rectangular 8-node plane stress elements	CQ16M
Lintels	3 node beam elements	CL9BE
Foundation	3+3 node 2 line interface elements	CL12I

Table 9: Element types used in numerical model

Interface properties; tangential stiffness.

Since the measured horizontal building movements are less accurate than the vertical displacements, it was not possible to determine the tangential stiffness of the interface elements. In a parameter study the tangential stiffness was varied, but the influence on the horizontal displacements of the considered points was small. This is elaborated in Appendix 3. At high tangential interface stiffness the horizontal movements of the considered points were similar to the ones at low tangential interface stiffness, while the horizontal displacement at the bottom side of the foundation layer of the facade did increase. This implies that the applied horizontal displacements can't be transferred from the bottom of the façade to the top, which is caused by the large amount of perforation of the façade. The horizontal displacements concentrate in the foundation layer of the facades.

It is chosen to adopt the horizontal interface stiffness as determined by Netzel (2009). This interface conditions allow some horizontal displacement to be transferred from the soil to the building. The horizontal displacements that can be transferred to the building is limited since at high horizontal ground movement the soil will undergo plastic deformation and will no longer affect the building; therefore the properties as given by Netzel (2009) are used for the foundation interface. This rough foundation interface properties are given in Table 10 and are illustrated in Figure 30.

Rough interface properties		
Normal stiffness	k_n	varying
Tangential stiffness	k_s	$2 \cdot 10^4 \text{ (N/m}^3\text{)}$
Tensile strength	$f_{t,i}$	$0 \text{ (N/m}^3\text{)}$
Cohesion	c	$0 \text{ (N/m}^3\text{)}$
Tangent of friction angle	$\text{tg } \phi$	0.57 (-)
Tangent of dilatancy angle	$\text{tg } \psi$	0 (-)

Table 10: Properties rough interface

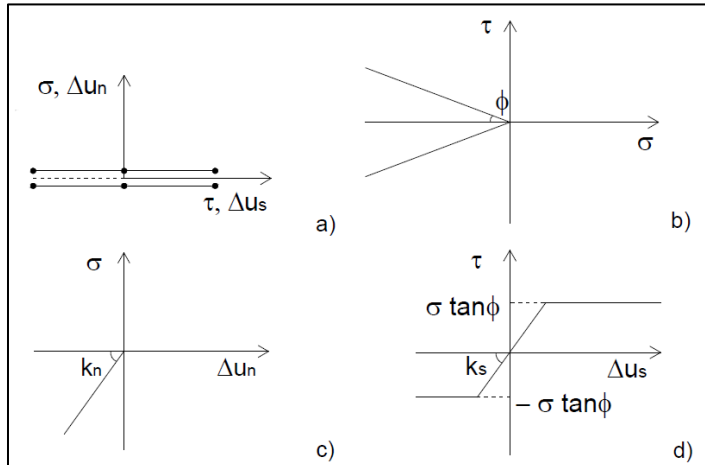


Figure 30: Rough interface: (a) element topology; (b) Coulomb friction criterion; (c) normal behaviour with zero tension criterion; (d) Coulomb criterion in the tangential direction. Giardina et al. (2010)

Loads

The self-weight of the facades is applied in the model.

The settlement trough and horizontal displacement profile of the first sand layer as were determined in week 3 of the case study are applied to the lower side of the interface. The maximum settlement is applied exactly where the tunnel centreline is, see Figure 28. The stiffness of the interface elements in normal and tangential direction determines the amount of displacements that are given through to the facades. The applied displacement profile is given in Figure 31.

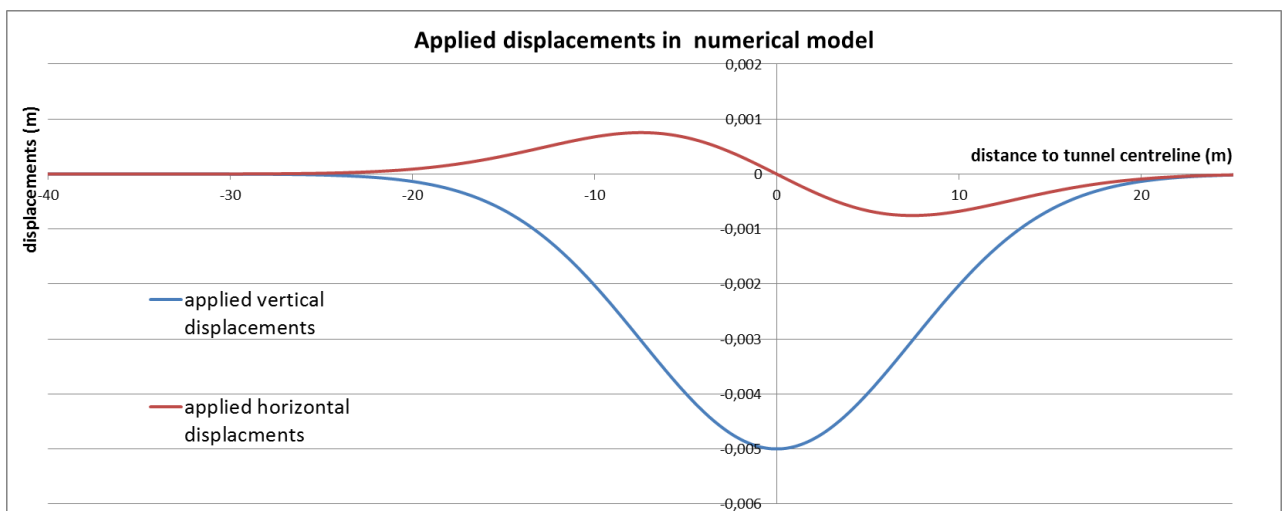


Figure 31: Horizontal and vertical displacement profile applied to the bottom side of the interface

Boundary conditions

In a first numerical model the two facades on the right side of the model in Figure 28 were not modelled because it was uncertain whether these two facades were attached to the rest of the facades. It appeared that if the two facades were left out, no good match could be found with the measurements because right side of the numerical model side showed too large vertical displacements. With the 2 extra facades the model matched the results. From this it can be concluded that the numerical model is sensitive to the applied boundary conditions.

4.2 Calibrating model parameters to field measurements

In this paragraph the numerical model will be tweaked to match building movements obtained in the case study under the applied settlements. The exact values for the masonry's Young's modulus and the stiffness of the foundation, represented by the interface elements, are not known. With the applied loads, the results should match the following boundary conditions:

1. The displacements of the points in the numerical model should match the displacement of the prisms as was measured by the monitoring system.
2. The applied trough should not cause cracks in the facades, since these were not found after passing of the TBM

Linear analyses are used to verify the first boundary condition at different Young's moduli. The second boundary condition is examined with nonlinear analyses.

4.2.2 Linear analyses

Using linear analyses the façade model parameters are tuned to match the measured vertical displacements. The displacements of the points matching the position of the prisms are extracted from the model. In nonlinear analyses the second boundary condition is checked, this is treated in the Paragraph 4.2.3.

The parameters to be studied are:

- Young's modulus masonry (E_m)
- Normal stiffness interface elements (k_n)

The properties of the model for the linear analyses are given in Table 11.

Material	Characteristic	Value
Masonry	Young's modulus	$E_m = \text{varying}$
	Density	$\rho_m = 1800 \text{ kg/m}^3$
	Poisson ratio	$\nu_m = 0.3 (-)$
Steel lintels	Young's modulus	$E_s = 210.000 \text{ N/mm}^2$
	Density	$\rho_m = 7800 \text{ kg/m}^3$
	Poisson ratio	$\nu_s = 0.2 (-)$
Interface	Normal stiffness	$k_n = \text{varying}$
	Tangential stiffness	$k_s = 2 \cdot 10^4 \text{ (N/m}^3\text{)}$

Table 11: Material and interface properties in the linear model

The intermediate steps in the tweaking process are given in Appendix 3. Only the final results will be discussed in this paragraph.

At different Young's moduli different normal interface stiffness is applicable. The stiffness's of interface that match the measured vertical displacements of the case study are summed in Table 12.

Young's Modulus	interface normal stiffness (k_n)	interface tangential stiffness (k_s)
$E_m = 500 \text{ N/mm}^2$	$5 \cdot 10^6 \text{ N/m}^3$	$2 \cdot 10^4 \text{ N/m}^3$
$E_m = 1000 \text{ N/mm}^2$	$1.2 \cdot 10^7 \text{ N/m}^3$	$2 \cdot 10^4 \text{ N/m}^3$
$E_m = 2000 \text{ N/mm}^2$	$2 \cdot 10^7 \text{ N/m}^3$	$2 \cdot 10^4 \text{ N/m}^3$
$E_m = 5000 \text{ N/mm}^2$	$5 \cdot 10^7 \text{ N/m}^3$	$2 \cdot 10^4 \text{ N/m}^3$
$E_m = 10000 \text{ N/mm}^2$	$1 \cdot 10^8 \text{ N/m}^3$	$2 \cdot 10^4 \text{ N/m}^3$

Table 12: Stiffness of the interface elements at different Young's moduli that match the measurements

The result of the Young's moduli of 500 – 2000 N/mm^2 are given in Figure 32. Not all points are visible because they overlap each other.

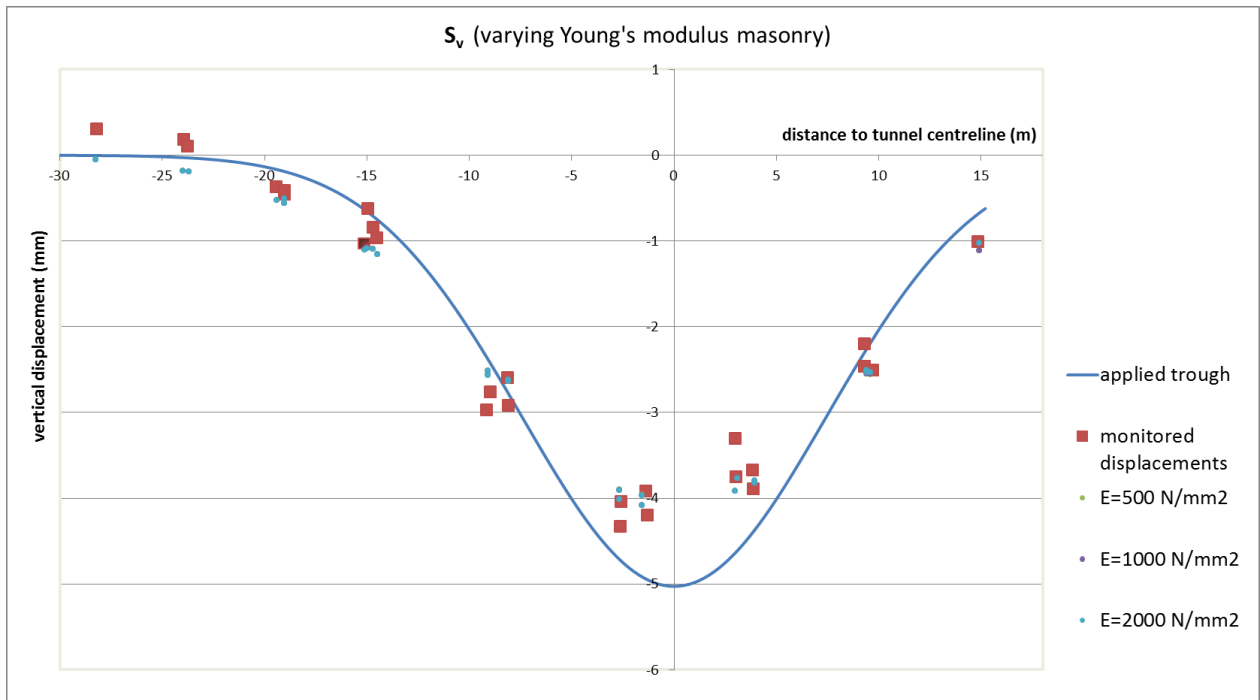


Figure 32: Vertical displacements of the building at different Young's moduli, smooth interface; linear analyses

The figure shows that the results of the façade model show good comparison with the field measurements. The first boundary condition has been fulfilled.

The horizontal displacements at different Young's moduli are given in Figure 33.

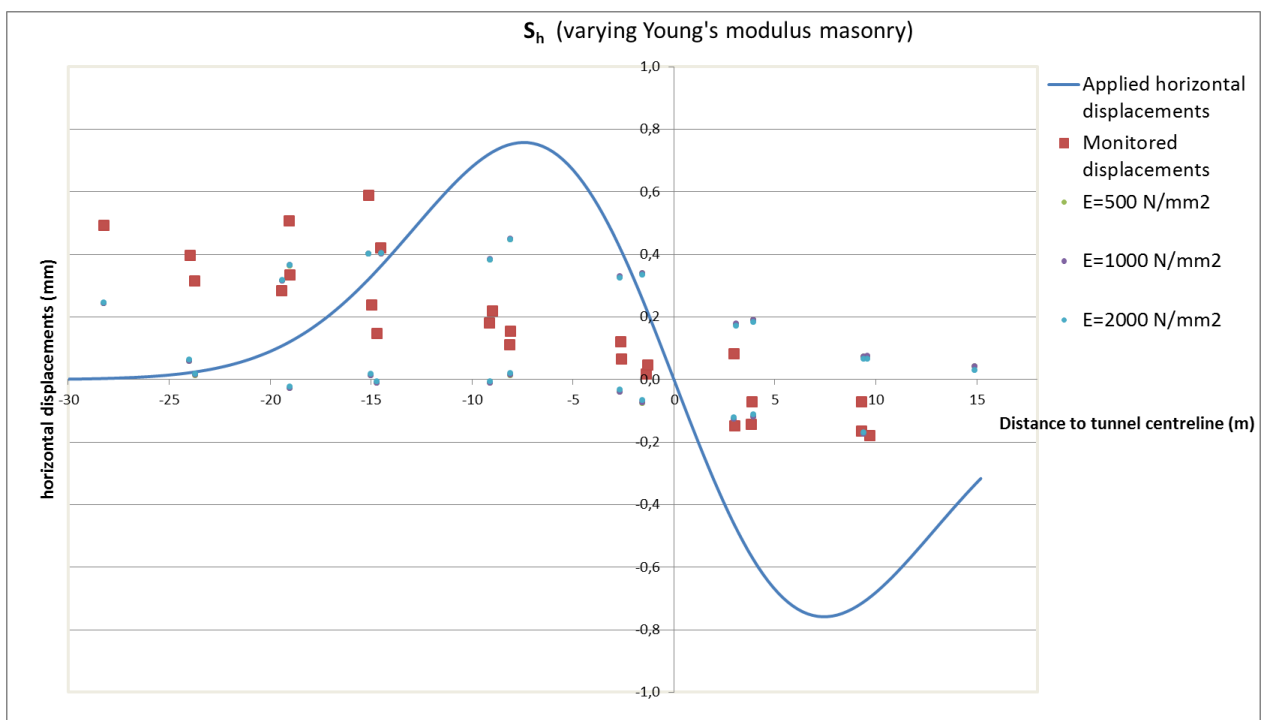


Figure 33: Horizontal displacements of the model at different Young's moduli in linear analyses, monitored displacements included

The figure shows that the Young's moduli only slightly influence the horizontal displacement a lot. Therefore no conclusion for the appropriate Young's modulus can be drawn on the basis of the horizontal displacements.

4.2.3 Nonlinear analysis

Nonlinear analyses are used to verify whether the second boundary condition can be met as well, i.e. the model should not show cracks. The properties used in the linear analyses for different Young's moduli will be checked with a nonlinear masonry model. In this case a smeared crack model with exponential tension softening will be used. The nonlinear masonry model will reveal whether the facades crack under the applied settlements.

Before starting the analyses an important assumption is made. It is uncertain what the tensile strength of the masonry in the Daniel Stalpertstraat is. Masonry properties differ per houses and no conclusion can be drawn from the case study upon the tensile strength to be applied. Since the range of tensile strength can reach from 0.1 to 1 N/mm² (Hendriks et al. (1995)), a value of 0.3 N/mm² is chosen; this value used by Witteveen+Bos as well. If the whole range of the tensile strengths should be examined, a too wide range of possibilities has to be considered. The fracture energy is kept at 50 N/m. It is emphasized that the choice of tensile strength strongly influences the cracking of the model!

The nonlinear masonry properties are listed in Table 13.

Property	value
Young's modulus	$E_m = \text{varying}$
Poisson ratio	$\nu = 0.3$
Density	$\rho_m = 1800 \text{ kg/m}^3$
Tensile strength	$\epsilon_t = 0.3 \text{ N/mm}^2$
Fracture energy	$E_f = 50 \text{ N/m}$

Table 13: nonlinear masonry properties

The cracking of the façade at different Young's moduli is listed in Table 14. It appears that, based on the second boundary condition and under the assumed tensile strength, the Young's modulus of the masonry should be below $E_m = 2000 \text{ N/mm}^2$. The crack pattern can be found in Appendix 3.

Young's modulus	cracking
$E_m = 500 \text{ N/mm}^2$	No cracking
$E_m = 1000 \text{ N/mm}^2$	No cracking
$E_m = 2000 \text{ N/mm}^2$	Cracked
$E_m = 3000 \text{ N/mm}^2$	Cracked
$E_m = 5000 \text{ N/mm}^2$	Cracked
$E_m = 10000 \text{ N/mm}^2$	Cracked

Table 14: amount of cracking at different Young's moduli, relative to $E_m = 3000 \text{ N/mm}^2$

As was concluded in the case study, the houses in the Daniel Stalpertstraat have settled even more over time than assumed in this numerical model due the filling of the mitigation measures pit (Paragraph 3.2.1). This extra settlement did not result in damage either, so the value of E_m should be well below $E_m = 2000 \text{ N/mm}^2$. Therefore a Young's modulus of $E_m = 1000 \text{ N/mm}^2$ is chosen, which is within the limits determined by TNO (according to Hendriks et al. (1995), E_m should be between 1000 N/mm^2 and 10000 N/mm^2).

It is emphasized that these conclusions have been based on the assumption of the maximum tensile strength and the fracture energy; at higher tensile strength a higher Young's modulus can be applied. This will not be further elaborated in this thesis.

The properties of the masonry and the interface representing the foundation are listed in Table 15. These will be used for further analyses.

Young's modulus	Foundation interface normal stiffness (k_n)	foundation interface tangential stiffness (k_s)
$E_m = 1000 \text{ N/mm}^2$	$1.2 \cdot 10^7 \text{ N/m}^3$	$2 \cdot 10^4 \text{ N/m}^3$

Table 15: properties numerical model to be applied

The parameters found for the foundation interface can be checked by calculating the stiffness of the piles. This calculation can be found in Appendix 5. The results are given in the table.

Vertical stiffness wooden piles	Horizontal stiffness wooden piles
$4.3 \cdot 10^7 \text{ N/m}^2$	$2.08 \cdot 10^3 \text{ N/m}^2$

Table 16: calculated pile stiffness's

The order of magnitude of the vertical stiffness is good while the horizontal stiffness is overestimated. The stiffness of the foundation will not be adapted in further analyses.

4.3 Deformation of the numerical model

The data from the numerical model and the field data is used to calculate the bending (curvature) and shear distortion of the facades in the numerical model and case study. If both match, this would be another verification of the properties used in the numerical model. The deformation is compared to the characteristics of the applied trough.

For the trough characteristics the transversal settlement trough is used that is applied in the numerical model (1st sand layer). The first and second derivative of the vertical displacements can be determined, this is elaborated in Appendix 6.

The first derivative of the settlement trough is related to shear distortion (γ) of the facades, the second derivative, or curvature, to bending deformation (κ).

The curvature and shear distortion of the facades are determined by the displacements of the monitored points in the façade and the according points in the different models. The different points are positioned in a more or less rectangular pattern, as can be seen in Figure 28. The curvature and shear distortion of the facades can be extracted from the displacements of the 4 corners of the rectangles.

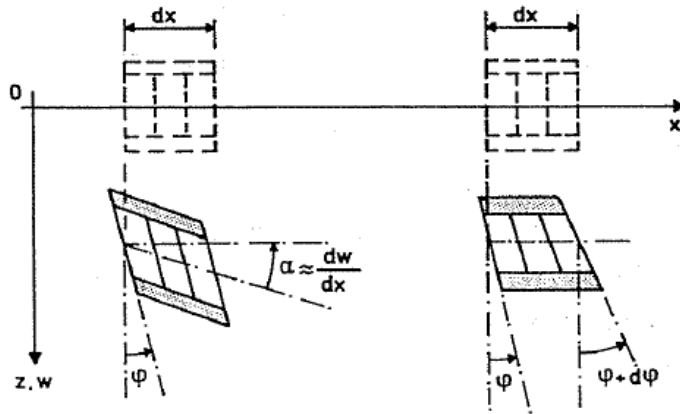


Figure 34: Shear distortion and curvature of a facade

The shear distortion and curvature can be written as:

$$\gamma = \frac{dw}{dx} + \varphi \quad (18)$$

$$k = \frac{d\varphi}{dx} \quad (19)$$

The curvature and shear distortion of the facades has been determined for every façade, for both the linear numerical model and field data. The results are given in Figure 35 and Figure 36. Also the curvature and shear distortion of the measurements have been put in the figures as well.

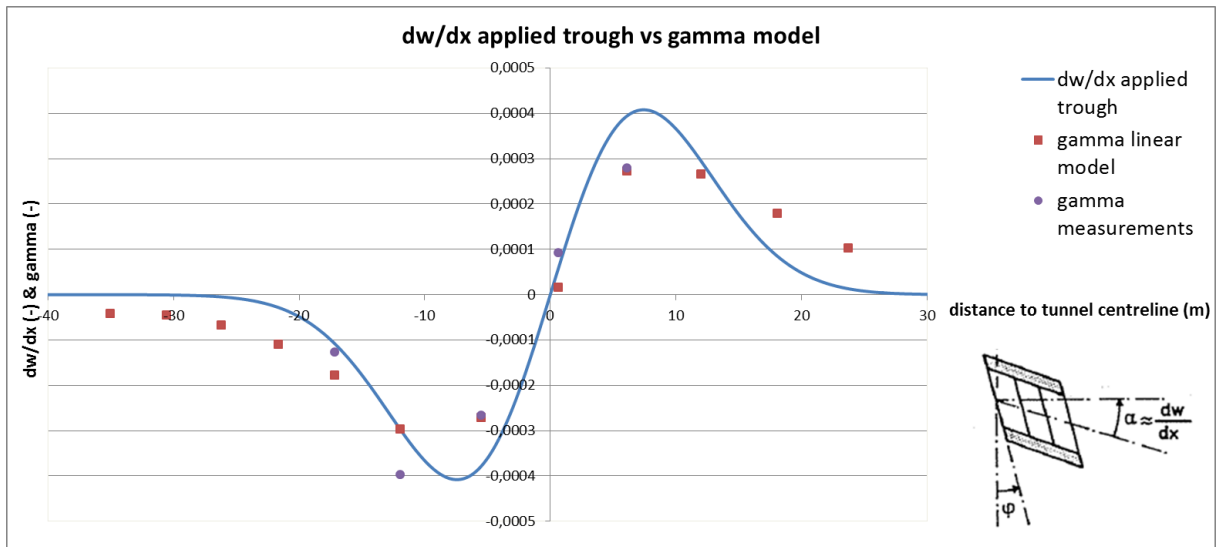


Figure 35: Shear distortion (gamma) of the numerical model (linear and nonlinear) and the measurements compared to the first derivative of the applied trough

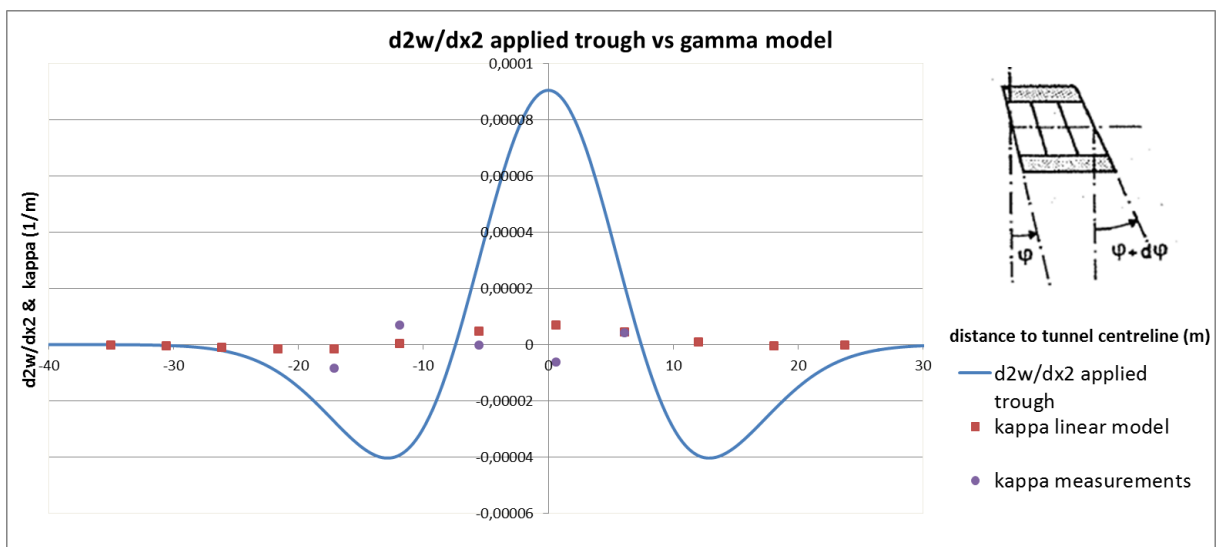


Figure 36: Curvature (kappa) of the numerical model (linear and nonlinear) and the measurements compared to the second derivative of the applied trough

In the figures it can be found that the shear distortion shows good comparison with the first derivative of the applied settlement trough, although the maximum value is lower than the first derivative of the applied trough. The curvature of the facades however is very low compared to the curvature of the applied trough. The measurements and results of the numerical model match each other well for the shear distortion, but less for the curvature. This can be blamed on the less accurate measurements of the horizontal displacements. Therefore it can be concluded that the numerical model is consistent with the measurements of the case study.

It can be concluded that the facades show low curvature and high distortion; the deflection of the facades are mainly caused by shear deformation. The many openings in the façade lower the shear stiffness of the façade significantly which results in dominant shear deformation.

4.4 Increase of settlement trough by extrapolation

The soil settlements obtained in the case study did not cause damage to the facades. In further analyses the behaviour of the model a larger settlements will be examined. With the results of the case study and numerical analyses performed in the SRA of the Daniel Stalpertstraat by Witteveen+Bos, an extrapolation of the soil settlements is made. This interpolation provides the settlement trough at higher volume loss ratio.

The calculation of the increased troughs can be found in Appendix 2. The trough parameters according to volume loss are given in Figure 37.

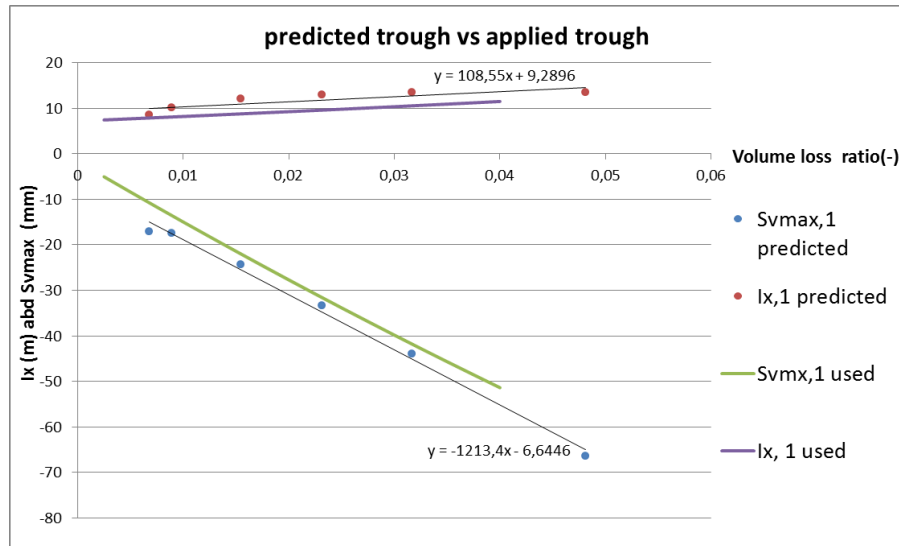


Figure 37: The predicted (in dots) against the applied (used) settlements of the 1st sand layer

Volume loss V_L (%)	Maximum settlement $S_{v,max}$ (mm)	Inflection point distance i_x (m)
0,23	5	7,435
0,49	10	7,810
0,78	15	8,200
1,09	20	8,610
1,43	25	9,020
1,80	30	9,450
2,20	35	9,900
2,64	40	10,370
3,10	45	10,850
3,61	50	11,360

Table 17: Characteristics of the applied troughs

The figure shows that the numerical model overestimates the settlements and overestimates the width of the settlement trough, if an extrapolation of the field data is used. This could be the results of the soil-structure interaction, but this is not further examined in this thesis. From the predictions it can be concluded that the calculation of settlement troughs by Peck (1969) are not correct in this situation, since the troughs do not have a constant inflection point distance as Peck assumes. At greater volume losses the inflection point distance increases.

4.5 Conclusions and recommendations

In this Paragraph the most important conclusions found in the tweaking of the model are listed. Also some recommendations are provided.

Conclusions:

- The numerical model with Young's modulus of the masonry of 1000N/mm^2 and a normal interface stiffness of $1.2 \cdot 10^7 \text{ N/m}^3$ match the measured movements of the case study well; at these values the numerical model does not crack either. The cracking however strongly depends on the assumed tensile strength.
- No conclusion can be drawn about the horizontal interface stiffness of the model and the horizontal soil movements to be applied; the measurements of horizontal building movements were not accurate enough.
- The numerical model is sensitive to boundary conditions.
- Due to the great amount of perforation of the façade the applied horizontal soil displacements cannot be transferred from the bottom to the top of the facades; the horizontal displacements concentrate in the bottom part of the façade.
- Due to the high perforation of the facades the shear stiffness of the facades low; shear distortion dominates the deflection of the facades. The curvature of the facades is low. This was also found in the measurements and is a verification of the correctness of the numerical model.
- The increased settlement trough, found by extrapolation of the field data, has an increasing inflection point distance at increasing volume loss.

Recommendations:

- The numerical model properties found are based on the assumed tensile strength of the masonry. This strength is highly uncertain; for a more reliable model the masonry properties in the Daniel Stalpertstraat should be examined.
- The increased settlement troughs are based on field data, extrapolated using numerical analyses. It is useful to measure the soil movement at deeper soil layers in future projects to verify the numerical analyses. This should result in a more reliable soil settlement prediction.

5. Linear analyses of increased settlements

In this chapter the linear numerical model of Chapter 4 will be used to examine its behaviour at increased settlements. First the influence of the increased settlement in the ratio between curvature and shear distortion is examined. Another goal of the linear analyses of the increased settlements is to determine how accurate damage prediction models are in terms of strains and deformation. In damage prediction models linear elasticity is assumed. If the calculation of strains and deformation in damage prediction methods would be reliable, the calculations should match the linear numerical results.

The increased settlement troughs determined in Paragraph 4.4 will be used. The deformations and strains at larger settlements will be determined; a distinction is made between local and global behaviour. The deformations and strains will be compared to 4 proposed damage prediction models. Two of them are LTSM models, applicable for the global behaviour:

1. Standard LTSM model. The settlement profile applied in the numerical model is used for calculations.
2. LTSM model with adjusted LTSM ratio. The settlement profile applied in the numerical model is used for calculations.

Two models using conventional beam theory, based on the local behaviour of the numerical model, might be applicable as a damage prediction model as well:

3. Portal frame model.
4. Forget-Me-Not model.

Finally conclusions will be drawn on the applicability of each method based on the linear numerical model.

5.1 definition of local and global behaviour

During the linear analyses it became clear that a distinction should be made between the local and global behaviour of the numerical model. Only with the distinction this deformations and strains found in the numerical model can be explained.

The global behaviour is the behaviour of the model as a whole; the facades all together are considered to be a perforated beam. This is comparable to the approach of the LTSM. In this case the maximum shear strains are situated at the neutral axis of the numerical model and the maximum bending strains at the bottom of the numerical model in the sagging zone. As will be explained in the Paragraph 5.2, the bending strains in the sagging zone are larger than the ones in the hogging zone. It will be shown however that the consideration of the global behaviour cannot explain the maximum tensile strain occurring in the numerical model; these can be found by considering the local behaviour. The consideration of the global behaviour is convenient to make a comparison with the LTSM, because in this method the facades are also considered as a beam.

The local behaviour of the facades is defined as the behaviour of one individual façade. An individual façade deforms like a portal frame under shear, introducing bending strains in the curved parts of the façade. This leads to the maximum tensile strain in the numerical model. This will be elaborated in Paragraph 5.3.

5.2 Global behaviour

In this paragraph the global behaviour of the numerical model will be studied. First the deformations of the model at increased settlements will be discussed. In the next step the global strains will be extracted from the model. The results of the linear numerical model will be compared to the results from two variants of the damage prediction method LTSM; both the standard LTSM the LTSM with adjusted E/G. These are chosen because the LTSM is based on the global behaviour of the facades. With the comparison the accuracy of the models can be determined in terms of strains and deformation.

5.2.1 Global deformations

The global deformations (shear distortion and bending) are again determined in the same way as was done in Paragraph 4.3. The maximum shear distortion (at inflection point distance) and the maximum curvature (at tunnel centreline) are given in Table 18.

V_L (%)	γ	κ	γ/κ
0,24	$2,71 \cdot 10^{-4}$	$6,78 \cdot 10^{-6}$	40,0
0,50	$5,22 \cdot 10^{-4}$	$1,33 \cdot 10^{-5}$	39,4
0,78	$7,65 \cdot 10^{-4}$	$1,95 \cdot 10^{-5}$	39,3
1,10	$9,90 \cdot 10^{-4}$	$2,54 \cdot 10^{-5}$	39,0
1,44	$1,22 \cdot 10^{-3}$	$3,10 \cdot 10^{-5}$	39,4
1,81	$1,44 \cdot 10^{-3}$	$3,64 \cdot 10^{-5}$	39,5
2,21	$1,66 \cdot 10^{-3}$	$4,21 \cdot 10^{-5}$	39,4

Table 18: Shear distortion, curvature and their ratio at increased settlements

The table shows that the increased settlements do not cause a change in the ratio between the bending and curvature; shear deformation remains the dominant deformation mode. The comparison of the curvature and shear distortion with the first and second derivative of the applied trough can be found in Figure 38 and Figure 39.

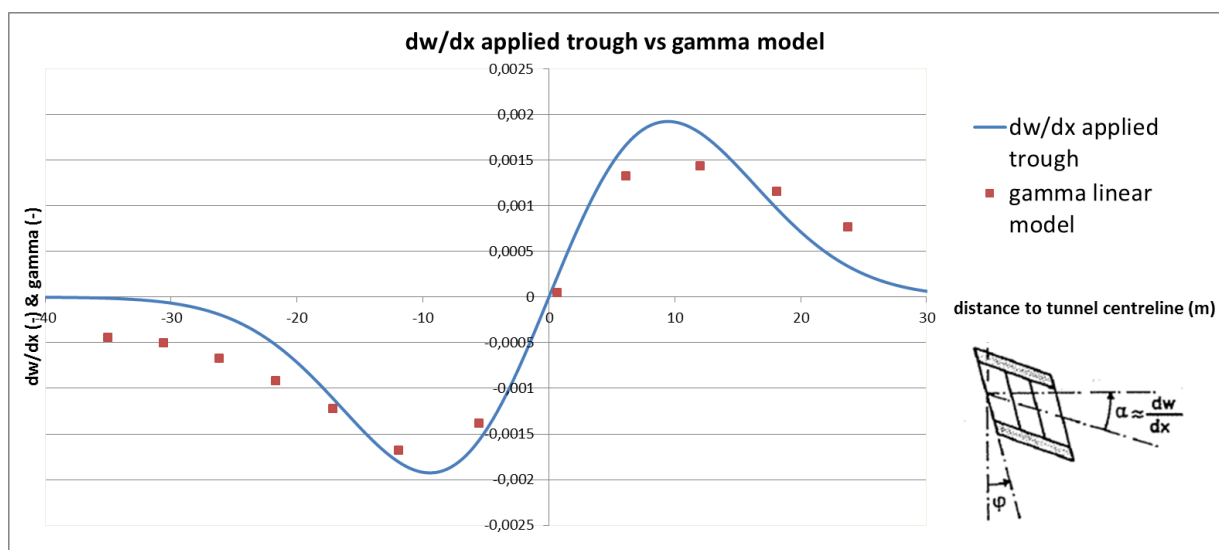


Figure 38: Shear distortion of the numerical model compared to the first derivative of the applied trough ($V_L=1.8\%$)

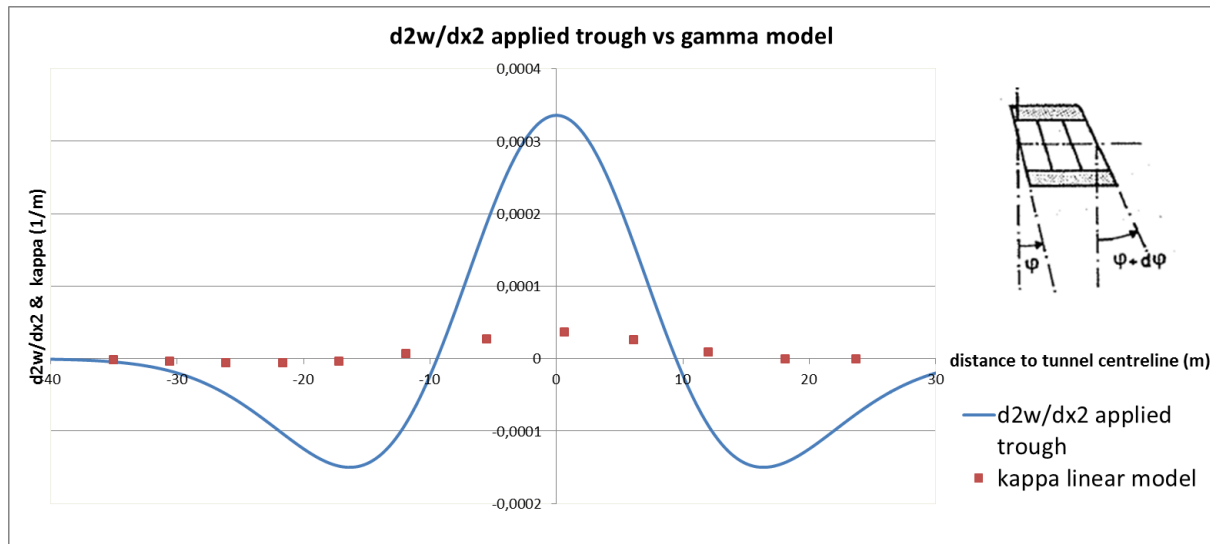


Figure 39: Curvature of the numerical model compared to the second derivative of the applied trough ($V_L=1.8\%$)

The figures show that also at increased settlements the shear distortion is comparable to the first derivative of the applied trough; curvature is much lower than the second derivative.

It can be concluded that increased settlements do not affect the deformation of the linear numerical model. The deflection is mainly caused by shear deformation.

5.2.2 Global strains

For the global behaviour the strains are extracted from the model. The maximum shear strains for the global behaviour are extracted from the neutral axis of the numerical model. It appeared that the maximum bending strains from the global behaviour are situated at the bottom of the numerical model in the sagging zone. The extraction lines of the strains are given in Figure 40.

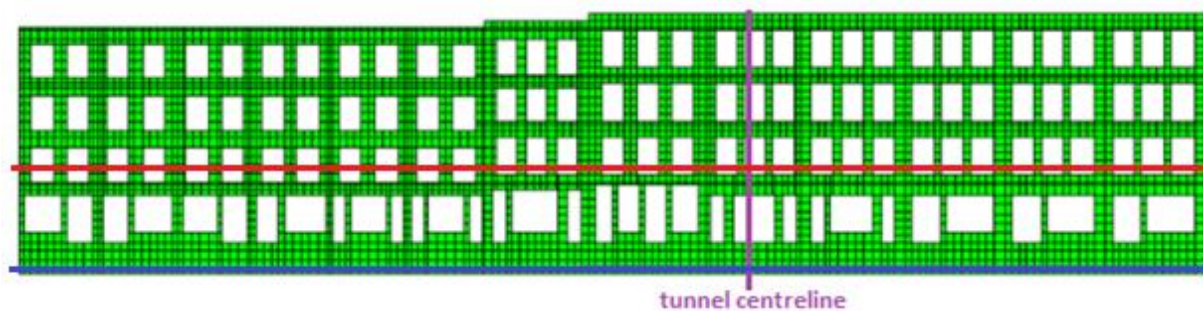


Figure 40: Extraction line of shear (red) and bending strains (blue)

The division of the shear and bending strains over the length of the model is given in Figure 41. These strains are extracted at volume loss ratio of 1.4%.

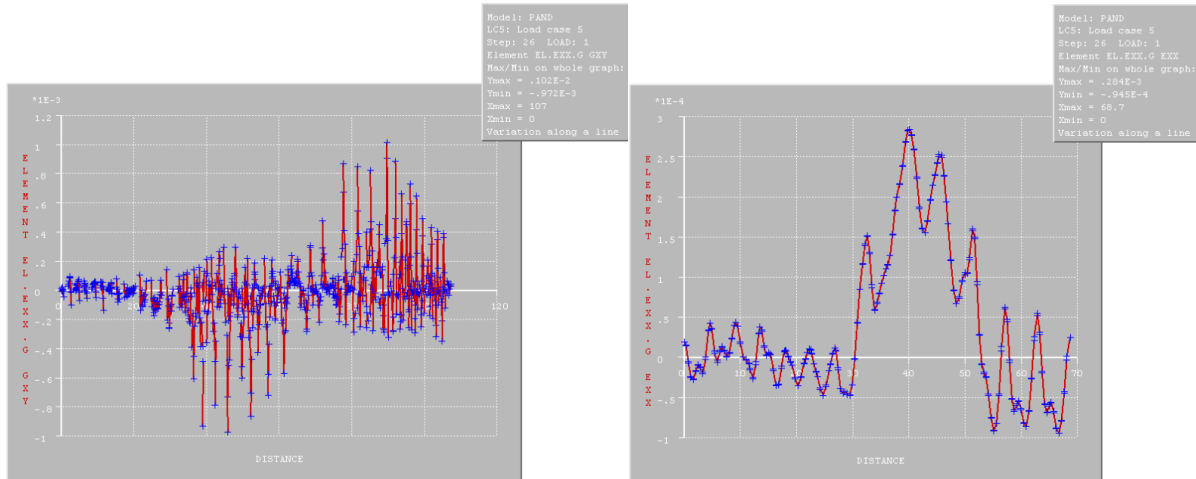


Figure 41: Division of shear strains in neutral axis (l) and bending strain at the bottom of the sagging zone (r) ($V_L=1.4\%$)

At the same lines also the principal strain (ϵ_p) are extracted at the considered volume loss ratios. The principal strains are the resultant strains of all combined strains. Also the maximum principal strains ($\epsilon_{p,max}$) in the whole numerical model have been determine. All are listed in Table 19.

V_L (%)	ϵ_b (%)	ϵ_d (%)	ϵ_p (%)	$\epsilon_{p,max}$ (%)
0,24	0,012	0,028	0,015	0,020
0,50	0,017	0,045	0,021	0,032
0,78	0,022	0,062	0,034	0,043
1,10	0,026	0,076	0,043	0,051
1,44	0,031	0,087	0,048	0,065
1,81	0,033	0,096	0,053	0,074

Table 19: Shear, bending and principal strains at considered positions compared to the maximum strains in the model.

The table shows that the shear strains are much larger than the bending strains in the global behaviour. At the neutral axis however, the principal strains are almost 50% lower than the shear or bending strains. This is caused by the dead weight of the facades; compression due to self-weight prestresses the elements. The prestressing increases the resistance to shearing. The self-weight can therefore be concluded to be important for the results in terms of strains.

The maximum principal strains however cannot be found when considering the global behaviour; the maximum principal strains are not concentrated in the neutral axis or the bottom of the facades. The principal strains found in the global behaviour are 25% lower than the maximum principal strain. The maximum principal strains can only be found by considering the local behaviour at the corners of openings in the façade; this will be treated in Paragraph 5.3.

5.2.3 Damage prediction models for global behaviour

The results of the linear global behaviour are used to make a comparison with the results of calculation in damage prediction models. With the damage prediction models deformations and strains in facades can be determined as well. The accuracy of the model in terms of strains and deformation can be evaluated.

The LTSM can be used for the simulation of this global behaviour. Two variants are used to examine their applicability in case of facades; the LTSM with E/G ratio of 2.6 and E/G ratio of 12.5. The results of the damage prediction methods in terms of strains and deformation are compared to the linear numerical results.

Standard LTSM (E/G=2.6)

The LTSM as described in Chapter 2 will be used to calculate the different strains and deformations due to the imposed settlements. From the calculations and in the numerical model it appeared that the sagging zone provides the governing strains. The facades that are examined have a total height of 14.75m and a length in the sagging zone of 14.86 to 18.9 m, giving a varying L/H ratio of 1.02 to 1.30. A uniform thickness of 0.22m is used for the calculations and an E/G ratio of 2.6. According to Figure 42 the bending strain should be governing in this situation. This is however not found in the global behaviour of the numerical model.

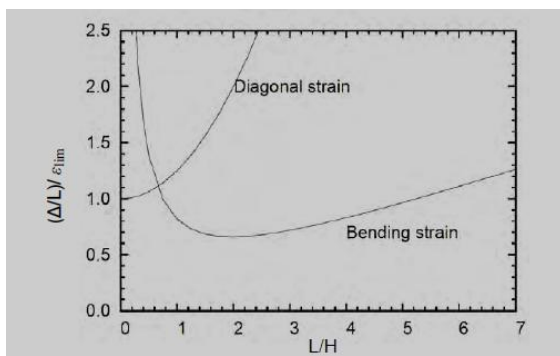


Figure 42: Dominant strains in the sagging zone at different L/H ratios, Burland et al (1974)

Comparison deformations

First the bending and shear behaviour of the LTSM is compared to the numerical results. The curvature and shear distortion of the LTSM beam can be derived from the Timoshenko beam theory, which is elaborated in Appendix 7. The results are presented in Figure 43 and Figure 44.

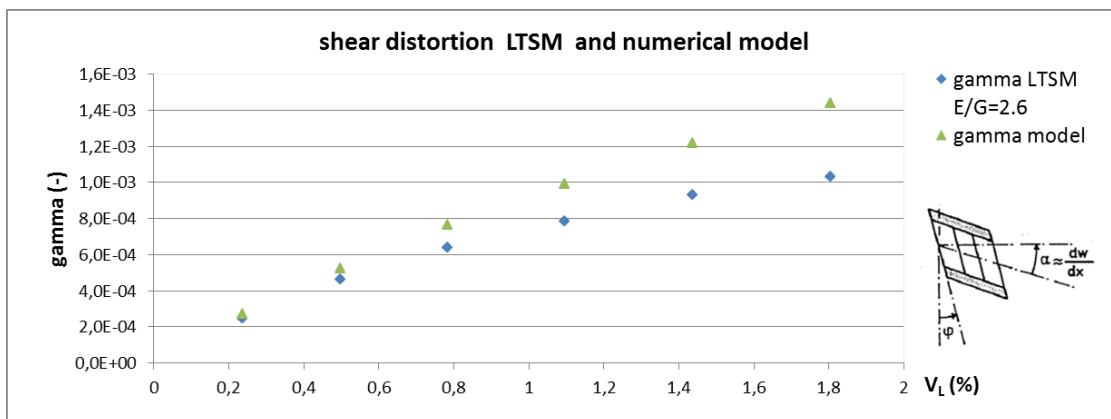


Figure 43: Shear distortion of the numerical model and the LTSM

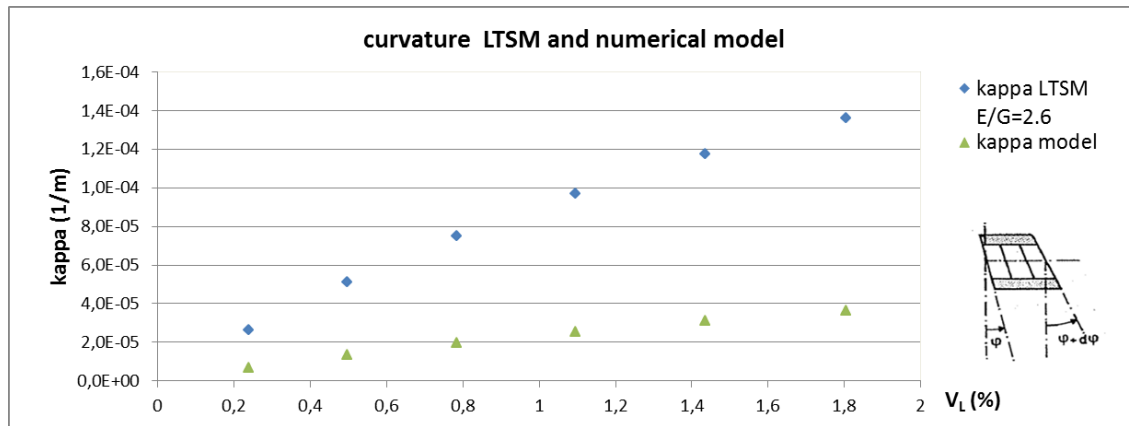


Figure 44: Curvature of the numerical model and the LTSM

The figures show that curvature of the facades is overestimated, which can be expected when one models a façade as a solid beam; the loss of shear stiffness due to opening is not taken into account. The shear distortion calculated at low volume loss ratio at first is in comparison with the distortion of the numerical model, but at higher volume losses the LTSM underestimates shear distortion. It can be concluded that the classical LTSM is not a good method to represent the deformation of the facades in the linear numerical model.

Comparison Strains

The results for the LTSM calculation of the strains according to the applied troughs from Figure 37 are given in Table 20. The formulas for the calculations can be found in Paragraph 2.3.

V_L (%)	E/G=2.6		Numerical model (global behaviour)	
	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
0,237	0,020	0,012	0,012	0,028
0,497	0,039	0,033	0,017	0,045
0,783	0,056	0,046	0,022	0,062
1,096	0,072	0,057	0,026	0,076
1,436	0,088	0,065	0,031	0,087
1,805	0,102	0,072	0,033	0,096

Table 20: strains due to the applied trough

The table shows that indeed the bending strains are the maximum strains, which was also found in Figure 42. In the numerical model however it was concluded that the model deforms in shear, which should lead to larger shear strains than bending strains in the LTSM. This was also obtained in the evaluation of the strains in the global behaviour of the numerical model. The results of the LTSM calculations are compared to bending and shear strains determined in the consideration of the global behaviour in Paragraph 4.2.2. The results are given in Figure 45.

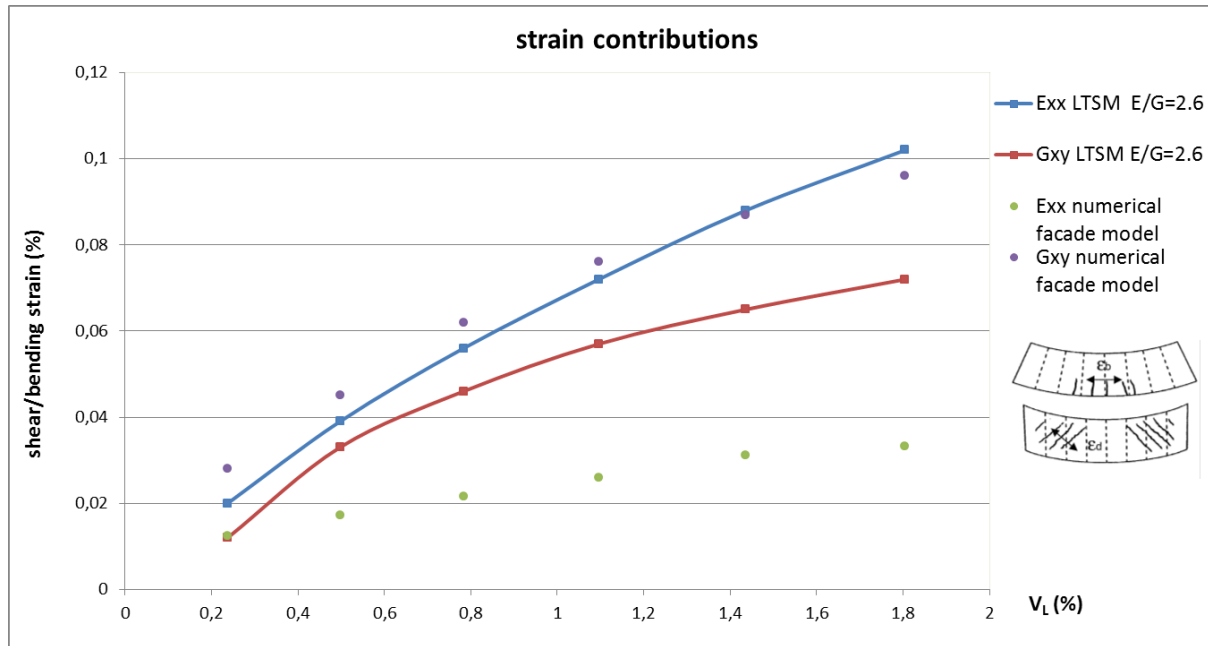


Figure 45: strains calculated using the LTSM with $E/G=2.6$ and the stains obtained from the numerical façade model.

The figure makes clear that the LTSM underestimates the shear strains with 30% and overestimates the bending strains with 120%, if the global behaviour is considered.

It can therefore be said that the LTSM in terms of strains does not represent the strains of the global behaviour of the linear numerical model.

Adjusted LTSM ($E/G=12.5$)

In case of portal frames the LTSM with an E/G ratio of 12.5 is often applied. This E/G ratio takes into account reduced shear stiffness of the portal frame compared to a solid beam. Because in the local behaviour of the numerical model it was obtained that an individual facade deforms like a portal frame, this E/G ratio is examined as well. The deformation of the local behaviour is treated in Paragraph 5.4. The results of this method are again compared to the results of the global behaviour of the numerical model

Comparison deformations

The curvature and shear distortion of the LTSM with $E/G=12.5$ are calculated. The results are given in Figure 46 and Figure 45.

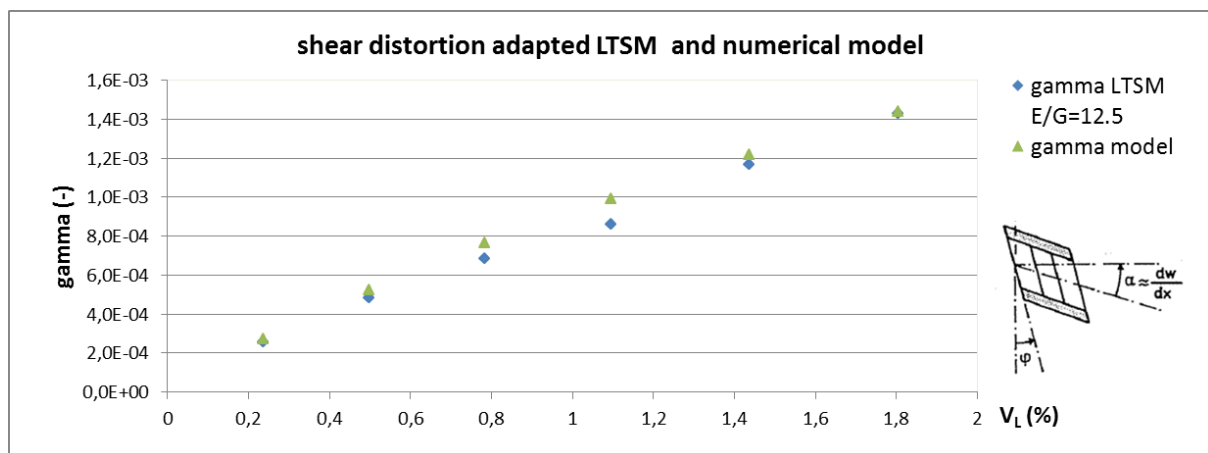


Figure 46: Shear distortion of the numerical model and the LTSM with $E/G=12.5$

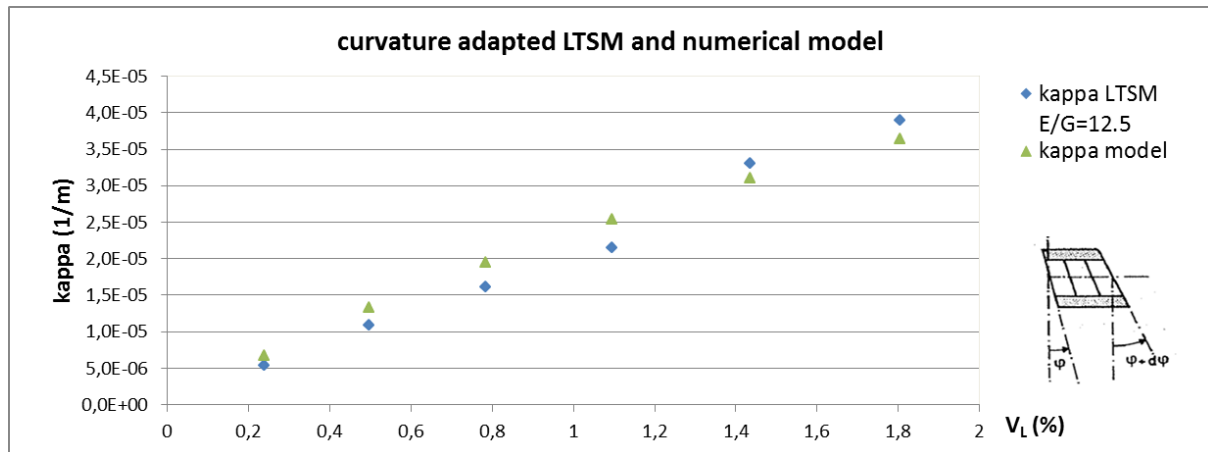


Figure 47: Curvature of the numerical model and the LTSM with $E/G=12.5$

The figures show that in the case of an E/G ratio of 12.5 the LTSM and the numerical show good comparison for both the shear distortion and the curvature. It can therefore be said that the LTSM with $E/G=12.5$ provides a good representation of the global behaviour of the facades in terms of curvature and shear distortion.

Strains

The strains were calculated with the adapted E/G ratio, these are listed in Table 21. In this table the strains are compared to the global results of the linear numerical model. The result is also illustrated in Figure 48.

V_L (%)	$E/G=12.5$		Numerical model (global behaviour)	
	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
0,237	0,003	0,024	0,012	0,028
0,497	0,007	0,036	0,017	0,045
0,783	0,010	0,050	0,022	0,062
1,096	0,016	0,065	0,026	0,076
1,436	0,020	0,077	0,031	0,087
1,805	0,024	0,088	0,033	0,096

Table 21: Strains determined with LTSM and a E/G ratio of 12.5 compared to the global numerical results

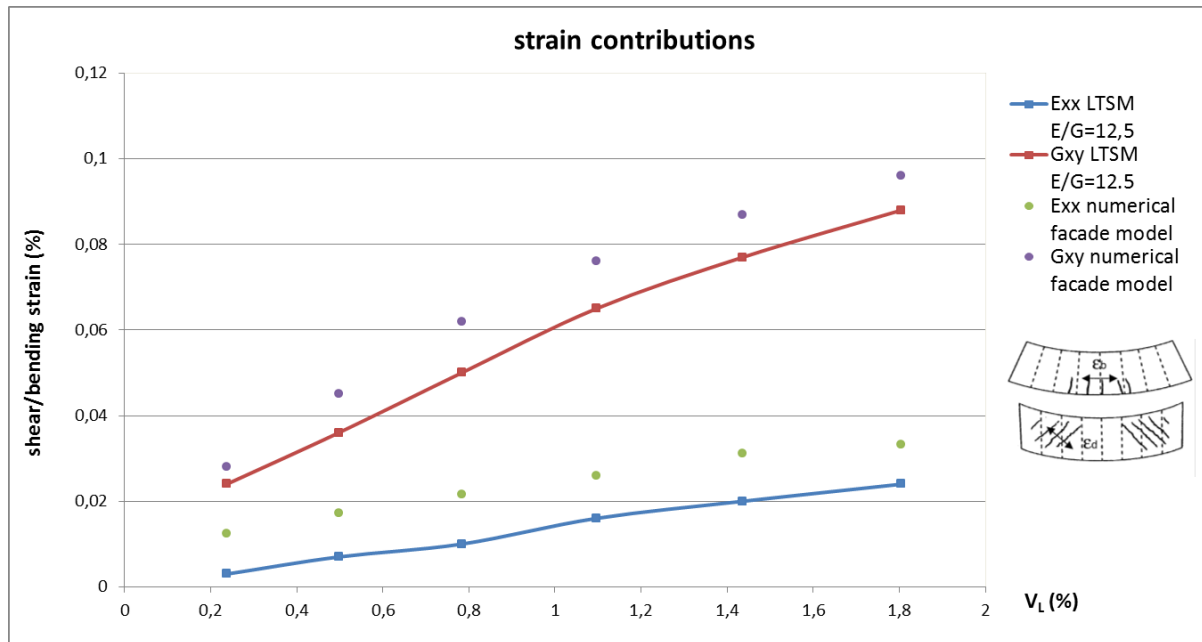


Figure 48: Strains contributions of the numerical model and the LTSM with $E/G=12.5$

The LTSM with $E/G=12.5$ determined the shear strains to be higher than the bending strains, which was also found in the global behaviour of the numerical model. The results of the values of the shear strains show good comparison with the results of the global strains of the numerical model, although the shear strains are slightly underestimated (about 10%). The bending strains are underestimated; the bending strains found in the numerical model are twice as high.

It can be concluded that the LTSM with $E/G=12.5$ represents the shear strains found in the global behaviour well, but the bending strain are underestimated.

5.3 Local behaviour

In the local behaviour an individual façade is considered. This consideration can explain the maximum principal strains occurring in the numerical model. The deformation and strains in the local behaviour will be extracted from the numerical model. Two possible damage prediction models are proposed that are based on this local behaviour; portal frame method and the Forget-Me-Not method. The models are based on conventional beam theory. With this models strain can be calculated. These strains can be compared to the strains found in the consideration of the local behaviour of the numerical model. The accuracy of the strains calculated with these models can be evaluated.

5.3.1 Local deformation

As was mentioned in the last paragraph, the global behaviour cannot explain the maximum strains found in the numerical model. The local behaviour of the façade is therefore considered.

The maximum tensile strains are found in the façade situated near inflection point distance. Once zooming in at this façade one can obtain that the individual façade deforms like a portal frame under shear, as described by Bouma (1989). The “columns” and “beams” of the façade are curved, as can be seen in Figure 49.

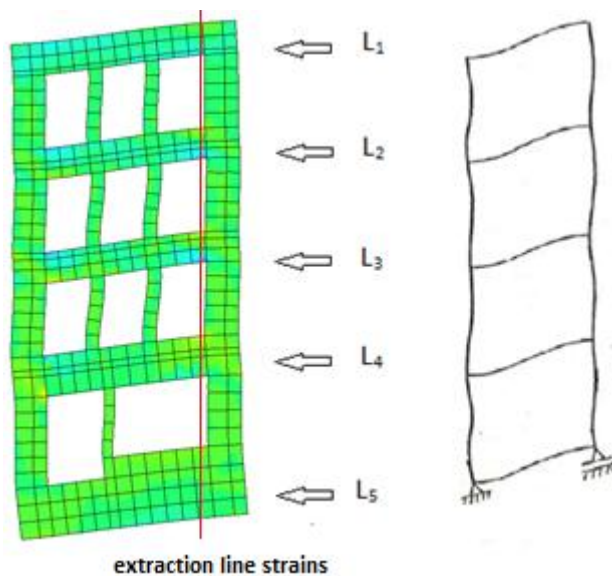


Figure 49: individual façade deforms like a portal frame as described by Bouma (1989)

5.3.2 Local strains

As can be seen from Figure 49, the maximum strains are concentrated at the corners of openings in the façades. The shear and bending strains are extracted from the model at the red line in Figure 49. The division of strain over the red line is given in Figure 50.

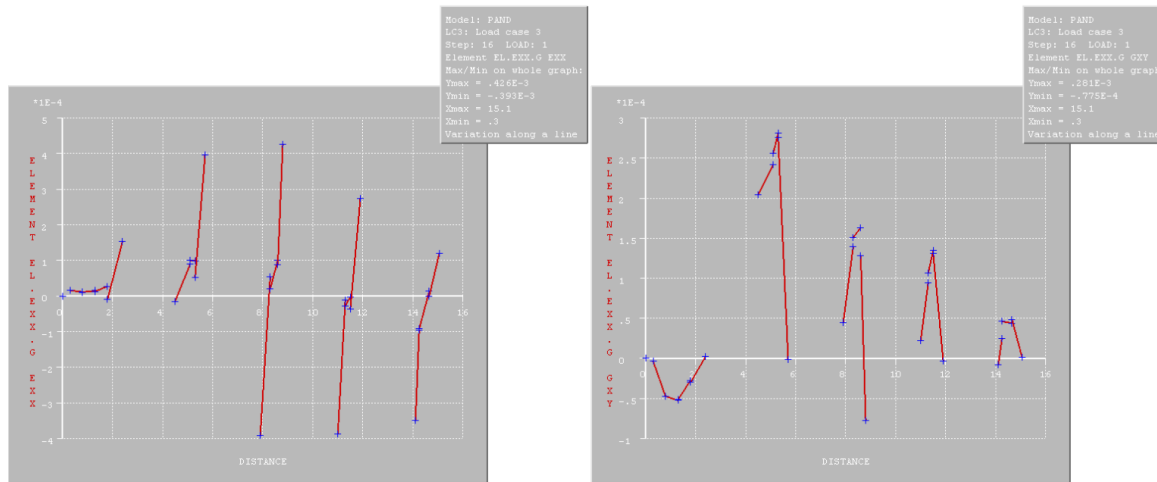


Figure 50: The bending (l) and shear strains (r) in the governing facade near the corners of the openings

The figures show that near the windows the bending strains are the maximum strain in this façade. The bending strains linearly increase from top to bottom at every “beam”, as is usual in beam theory. The maximum bending and shear strain found in the local behaviour are listed in Table 22. The beams are numbered from top to bottom as can be found in Figure 49.

V_L (%)	0.237		0.497		0.783		1.096		1.436		1.805	
Beam nr	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
L1	0,007	0,007	0,01	0,009	0,013	0,01	0,017	0,013	0,018	0,016	0,019	0,018
L2	0,012	0,012	0,021	0,015	0,028	0,022	0,038	0,028	0,04	0,032	0,046	0,039
L3	0,017	0,016	0,032	0,017	0,043	0,03	0,051	0,038	0,065	0,041	0,074	0,05
L4	0,017	0,02	0,03	0,024	0,041	0,03	0,049	0,037	0,059	0,041	0,066	0,047
L5	0,005	0,015	0,008	0,014	0,015	0,012	0,021	0,012	0,020	0,012	0,029	0,012

Table 22: Bending and shear strains in the governing facade

At all individual beams the bending strains are the maximum strains, except for the beam L5. The maximum bending strains provide the maximum principal strain in the local behaviour. The principal strains at beam L3 are equal to the maximum principal strain in the numerical model. This can be found in Table 23.

V_L (%)	ϵ_b (%)	ϵ_d (%)	ϵ_p (%)	$\epsilon_{p, \max}$ (%)
0,24	0,017	0,016	0,017	0,020
0,50	0,032	0,017	0,032	0,032
0,78	0,043	0,030	0,043	0,043
1,10	0,051	0,038	0,051	0,051
1,44	0,065	0,041	0,065	0,065
1,81	0,074	0,050	0,074	0,074

Table 23: shear, bending and principal strains in L3 and the maximum strain in the numerical model

The maximum strains in the model are found by considering the local behaviour of the numerical model. It can be concluded that despite dominant shear deformation, bending strains due to bending of parts of the façade provide the maximum principal strain in the numerical model.

5.3.3 Local damage prediction methods.

The analyses of the numerical model showed that not the global behaviour but bending of beams in local behaviour (individual façade) provides the governing strains in numerical model. This bending of beams results in local bending strains while the façade deforms in shear. In this paragraph two methods are proposed that are based on this local behaviour, and might have the ability to predict damage found in the numerical model; the portal frame method and the Forget-Me-Not method. The results of these models in terms of strains will be compared to the numerical results of the local behaviour.

Portal frame method

The local deformation looks like a portal frame subjected pure shear. A portal frame model of the façade, as described by Bouma (1989) could be a good way to model the local behaviour the facades. The possibility of using a portal frame model as representation of the local behaviour of the numerical model is examined.

Frame model

The model of the facades is given in Figure 51. Every column or beam in the model has a bending stiffness. This can be determined from the geometry of each part of the considered façade. Only the values for each part are given in Table 24, the calculation can be found in Appendix 7.

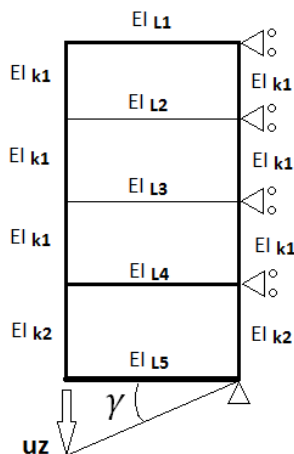


Figure 51: Portal frame model with different stiffness's of columns and beams

Beam/column	E (N/m ²)	I (m ⁴)	A (m ²)	L (m)
L1	1·10 ⁹	0,016	0,209	5.6
L2	1·10 ⁹	0,013	0,198	5.6
L3	1·10 ⁹	0,013	0,198	5.6
L4	1·10 ⁹	0,032	0,264	5.6
L5	1·10 ⁹	0,124	0,66	5.6
K1	1·10 ⁹	0,016	0,314	4.1
K2	1·10 ⁹	0,024	0,209	5.2

Table 24: Characteristics of the beams and columns of the portal frame model

For the length of each part the neutral axis of each column and beam is used. Also the self-weight of the beams and columns is taken into account in the calculation.

Strains

The applied γ in Figure 51 is the first derivative of the settlement trough, since this can be determined from the settlement through that is known at forehand. The bending moment, shear force and normal force in the beams and columns can be determined. Since the portal frame is a slender structure, the shear strain, normal strain and bending strain in every part of the portal frame

model can be determined through beam theory and Hooke's law. This is elaborated in Appendix 7; the results are listed in Table 25 and Table 26. In these tables a comparison is made with the values found in the numerical model (local behaviour). Two volume losses are considered in this table, $V_L=0.8\%$ and $V_L=1.8\%$. The results at other volume loss ratios can be found in appendix 7.

Beam nr	M (Nm)	N (N)	V (N)	Portal frame		Numerical model (local behaviour)	
				ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
L1	8860	4618	3219	0,029	0,010	0,013	0,01
L2	11259	-1347	4043	0,037	0,007	0,028	0,022
L3	11792	1424	4174	0,040	0,009	0,043	0,03
L4	25699	5901	9223	0,050	0,017	0,041	0,03
L5	26103	-10596	9524	0,014	0,002	0,015	0,012
K1	7975	5852	4432	0,027	0,011		
K2	26103	9524	10596	0,056	0,005		

Table 25: Forces, moments and strains in the beams at $V_L=0.8\%$

Beam nr	M (Nm)	N (N)	V (N)	Portal frame		Numerical model (local behaviour)	
				ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
L1	15427	8040	5605	0,050	0,017	0,019	0,018
L2	19728	-2346	7023	0,065	0,012	0,046	0,039
L3	20531	2480	7268	0,070	0,016	0,074	0,050
L4	44745	10274	16058	0,089	0,029	0,066	0,047
L5	45449	-18451	16582	0,025	0,003	0,029	0,012
K1	13877	10182	7712	0,047	0,019		
K2	45449	16582	18451	0,097	0,009		

Table 26: Forces, moments and strains in the beams at $V_L=1.4\%$

The tables show that the portal frame model does not show good comparison in terms of strains for most parts of the portal frame model, if compared to the numerical results of the local behaviour. Although the bending strains are the maximum strains in the portal frame model, the values for the strains differ much in most cases from the numerical results. The shear strains are underestimated. It should be remarked that the columns provide the governing strains in the portal frame model, while this is not obtained in the local behaviour of the numerical model; the maximum principal strains are found in the beams.

Only the results of the beams are considered now. If only the maximum strains in beams of the portal frame method are compared to the maximum principal strain in the numerical model, one can see that the bending strains of the portal frame are larger than found in the numerical model. This is provided in Table 27. The shear strains are underestimated in the portal frame method.

V_L (%)	Portal frame model		Numerical model (local behaviour)	
	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
0,237	0,018	0,006	0,017	0,02
0,497	0,036	0,012	0,032	0,024
0,783	0,051	0,017	0,043	0,03
1,096	0,065	0,026	0,051	0,038
1,436	0,077	0,021	0,065	0,041
1,805	0,089	0,029	0,074	0,05

Table 27: Maximum strain in the beams of the portal frame method (L3) compared to the maximum strains in the local behaviour of the numerical model

It can therefore be concluded that the portal frame model does not represent the local behaviour in term of strains.

Forget-Me-Not method (FMN method)

In the portal frame method it was concluded that the columns are the governing parts of the portal frame, while this was not obtained in the numerical model. A way to overcome this is to use one of the so called 'forget-me-not' equations, often used in structural mechanics (Hartsuiker (1999)). Again the results of the strains will be compared to the local behaviour of the numerical model.

The model is widely used in the strength and stability calculation and is given in Figure 52. In this model a vertical displacement (shear) is given to a bending beam, which could be a good way to model the behaviour of the considered façade.

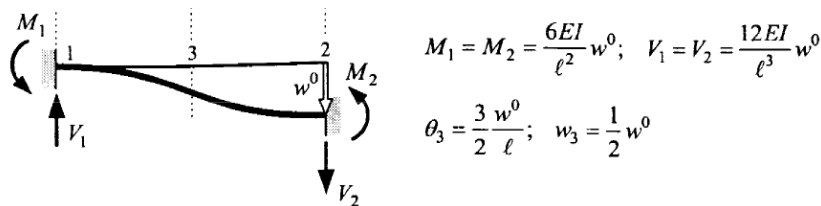


Figure 52: Forget-Me-Not model used (hartsuiker 1999)

In this model it is assumed that in case that the columns are infinitely stiff. The stiffness of the beam can be determined as the sum of the stiffness's of the individual beams in the local behaviour (Table 24). The equation for the shear force and bending moment turn into the following formulas:

$$M_1 = M_2 = 6 \frac{\sum_i EI_i}{l^2} w_0 ; \quad V_1 = V_2 = 12 \frac{\sum_i EI_i}{l^3} w_0 \quad (20)$$

In this case again the first derivative is applied to the beam model, only now the width of the columns is neglected and therefore the length of the beam reduces to 4.7m (the length of the beams between the columns). Calculation towards the strains can be found in Appendix 7.

Strains

The strains determined in this model are again compared to the results of the local numerical model. In this case the local maximum strains of the numerical model (Table 28) are compared to the maximum strains of the FMN method.

V_L (%)	FMN model		Numerical model (local behaviour)	
	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
0,237	0,033	0,006	0,017	0,02
0,497	0,063	0,010	0,032	0,024
0,783	0,090	0,015	0,043	0,03
1,096	0,114	0,019	0,051	0,038
1,436	0,136	0,024	0,065	0,041
1,805	0,156	0,027	0,074	0,05

Table 28: Strains determined in the FMN model compared to the local strains of the numerical model

The FMN method determines the bending strains to be the maximum strain, as was obtained in the local behaviour of the numerical model. The model however overestimates the values of the bending strains compared to the numerical model, the values are twice the value found in the numerical model. The shear strains are underestimated. The FMN model is not a good representation of the individual behaviour of the facades in the linear numerical model; the strains are overestimated.

5.4 Evaluation damage prediction methods

In Paragraph 5.3 the local and global behaviour of the numerical model have been studied and 4 damage prediction methods were examined for their applicability. In this paragraph the methods will be compared; the best model to represent the behaviour of the facades in the Daniel Stalpertstraat will be determined.

It is difficult to compare the local and global methods; the shear strains in the global behaviour are larger than the bending strains, but they do not provide the maximum principal strains in the numerical model. The shear strains however are larger than the maximum principal strain. The local behaviour of the model provides the principal strain, caused by bending strain (bending strains larger than shear strains).

For convenience all models will therefore be compared to the global and local behaviour in terms of deformation, strains and strain magnitude.

- Deformation consistent with global behaviour numerical model: the applied method implies that shear distortion is dominant over curvature, as was obtained in the global behaviour of the numerical model.
- Deformation consistent with local behaviour numerical model: the applied method implies that individual parts of the façade are curved, despite the fact the façade globally acts in shear deformation.
- Strains consistent with global behaviour numerical model: the applied method implies that shear strain are governing over bending strains, as was obtained in the global behaviour of the numerical model.
- Strains consistent with local behaviour numerical model: the applied method implies that bending strains are governing over shear strains due to bending of individual parts of the façade, as was obtained in the local behaviour of the numerical model.
- Strains magnitude: the strains calculated are equal to the values found in the numerical model. For the global methods comparison is made with the global strains, for local methods comparison is made with the local strains.

The results are listed in Table 29.

		LTSM E/G=2.6	LTSM E/G=12.5	Portal frame method	FMN model
Deformation	Consistent with global behaviour (<i>shear distortion over curvature</i>)	No	Yes	Yes (shear governing assumed in model)	Yes (shear governing assumed in model)
	Consistent with local behaviour (<i>act like portal frame/bending beams</i>)	No	Yes (with E/G=12.5 a portal frame is assumed)	Yes	Yes
Strain	Consistent with global behaviour (<i>shear over bending</i>)	No	Yes	No	No
	Consistent with local behaviour (<i>bending over shear</i>)	Yes, but bending in global sense	No	Yes	Yes

Strain magnitude	Calculated <i>bending</i> strains compared to numerical results	Overestimated (max value 300% of numerical value)	Good (max value 90% of numerical value)	Overestimated (max value 130% of numerical value)	Overestimated (max value 200% of numerical value)
	Calculated <i>shear</i> strains compared to numerical results	Underestimated (max value 75% of numerical value)	Underestimated (max value up to 50% of numerical value)	Underestimated (max value 60% of numerical value)	Underestimated (max value 50% of numerical value)

Table 29: different models compared to local and global behaviour of the numerical model

The table shows that none of the methods scored well on every point for the linear numerical model. The LTSM with $E/G=2.6$ scores worst of all; it does not deform as the numerical model does, and the strain contributions and values are not comparable to the numerical results either.

As mentioned before the portal frame method only does well if the results of the columns are neglected. Even then the strains are over- or underestimated. The FMN model does not include columns, but the method overestimates the bending strains with 100%; this is not representative if compared to the local behaviour of the model.

The LTSM with $E/G=12.5$ is the only method that at least comes near the results found in the numerical model. The deformation of this method is consistent with local and global deformation, and the strains are comparable with the numerical model in global sense. It is the best method of the four for the linear numerical model.

The maximum strains calculated with the LTSM are however larger than the maximum principal strain found in the numerical model. Nonlinear analyses should show the performance of the methods in terms of damage estimation.

5.5 Conclusions and recommendations linear model

The conclusion and recommendation of the linear model are listed below.

- A distinction between local and global behaviour of the linear numerical model had to be made to explain the results.
- In the global behaviour it was concluded that the ratio between shear and bending deformation did not change due to the imposed settlements. The magnitude of settlement does not affect the shear/bending ratio; shear deformation remains the dominant deformation mode. The ratio between shear distortion and curvature remains 40.
- In the global behaviour of the linear numerical model it was found that shear strains are larger than the bending strains. This is a result of the high shear distortion and low curvature of the numerical model. The strains in the global behaviour however do not provide the maximum principal strains in the numerical model. The principal strains in the global behaviour are 30% lower than the maximum principal strains in the numerical model.
- Self-weight of the facades is important; prestress due to self-weight increases the resistance against shear.
- The LTSM with E/G ratio of 12.5 shows good results if compared with the global behaviour of the linear numerical model. The curvature and shear distortion calculated with this adjusted LTSM are equal to the ones found in the numerical model. The shear strains are approximated with 90-95%. The LTSM with E/G=2.6 does not provide a good comparison for the deformations (too high curvature, too low shear distortion), and the bending strains are overestimated while the shear strains are underestimated.
- In the local behaviour it was obtained that an individual façade deforms like a portal frame under shear, introducing local bending strains in the corners of windows. In the local behaviour the bending strains are larger than the shear strains.
- Both the portal frame method and the “forget-me-not method” are no good models to calculate strains in the compared to the local behaviour. For the portal frame method the columns provide the maximum strains, while this is not obtained in the linear numerical model. The Forget-Me-Not method overestimates the strains with 100%, compared to the results of local behaviour of the linear numerical results.
- The LTSM with E/G=12.5 is decided to be the best method for damage prediction in the case of facades, if the linear model is considered.

6. Nonlinear analyses of increased settlements

In this chapter the nonlinear numerical model of Chapter 4 will be used to examine its behaviour at increased settlements. The damage prediction models in Chapter 5 did in most cases not show the same strain contributions as was found in the numerical model. With the nonlinear numerical model it can be checked that, despite the inaccuracy of the calculated strains, the predicted damage is consistent with the nonlinear analyses results.

The increased settlement troughs determined in 4.4 will be used. The cracking of the numerical model is examined and the influence of the cracks on the behaviour of the numerical model as well. This cracking (damage) will be compared to the results of the damage prediction method treated in Chapter 5.

The definition of the tensile strength has a great influence on the cracking of the model. The assumption of the tensile strength will be discussed first.

6.1 Assumptions and their influence on the results

As was already mentioned in Chapter 4, the tensile strength of the masonry of the houses in the Daniel Stalpertstraat is unknown. Also literature does not provide one single definition of the tensile strength of masonry. The properties of masonry depend on many factors; the kind of brick used, the kind of mortar used, the bond, the age, etc. According to Hendriks et al (1995) it can range from 0.1 up to 1 N/mm². At Witteveen+Bos the tensile strength is set at 0.3 N/mm²; this is also applied in the nonlinear analyses.

The influence of the chosen tensile strength is however great; a higher tensile strength leads to a higher allowable strain. Therefore at a higher volume loss a lower crack width will be found if higher tensile strength is used.

This does not provide a solid basis for the results of the nonlinear analyses. Performing nonlinear analyses could at least give an indication of the way the facades crack and if this influences the behaviour of the facades. Also the damage prediction methods can be tested for their applicability in the case that the assumed tensile strength is right. This should be kept in mind while reading this chapter.

For a more reliable nonlinear model the masonry at the Daniel Stalpertstraat should be tested for its properties.

In this report it is assumed that the nonlinear model is representative for the behaviour of the facades under increased settlements.

6.2 Cracking and post cracking behaviour

First the cracking of the façade is examined by performing nonlinear analyses. The results of the crack widths and the influence of cracking on the behaviour of the numerical model is studied.

6.2.1 Cracking and crack propagation

At increasing settlements the facades start to crack. The first small cracks occur at settlements of 15 mm, or a volume loss ratio $V_L=0.78\%$. The first cracks appear at the corners of the opening in the façade, which are weak spots within the facades. The reason for this cracking was already determined at the consideration of the local behaviour of the numerical model in Chapter 5.4; portal

frame deformation leads to local bending strains at the corners of the openings. The maximum crack width at increasing volume loss ratio is illustrated in Figure 53.

The maximum crack width is defined as the largest crack width found in the numerical model by multiplying the average crack strain in one element with the crack bandwidth.

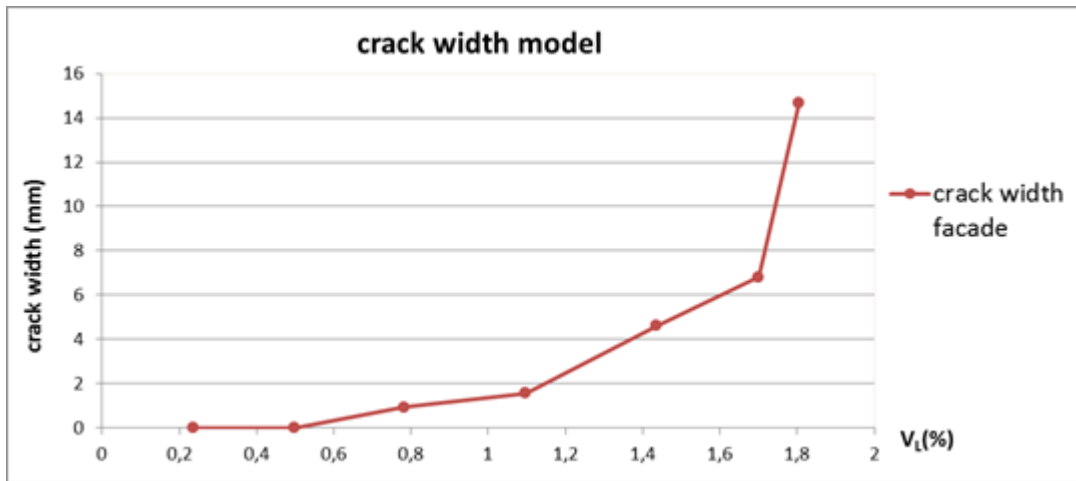


Figure 53: Maximum crack width in the model at increasing volume loss ratio

As the graph shows, the maximum crack width gradually increases until the $V_L=1.8\%$, at which a major crack is initiated.

According to Table 3 a structure suffers from structural damage at crack widths of $w_{\text{crack}} > 25\text{mm}$ (structural damage), but as Figure 57 shows, the façade is totally cracked from the bottom to almost the top of the model a volume loss ratio of 1.8%; structural failure has already occurred here.

The crack initiation and propagation are given in Figure 54 to Figure 58.

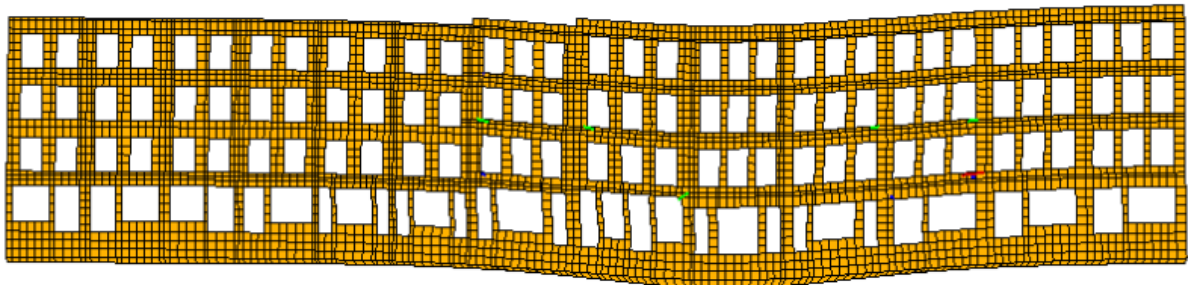


Figure 54: appearance of first cracks at $V_L=0.78\%$

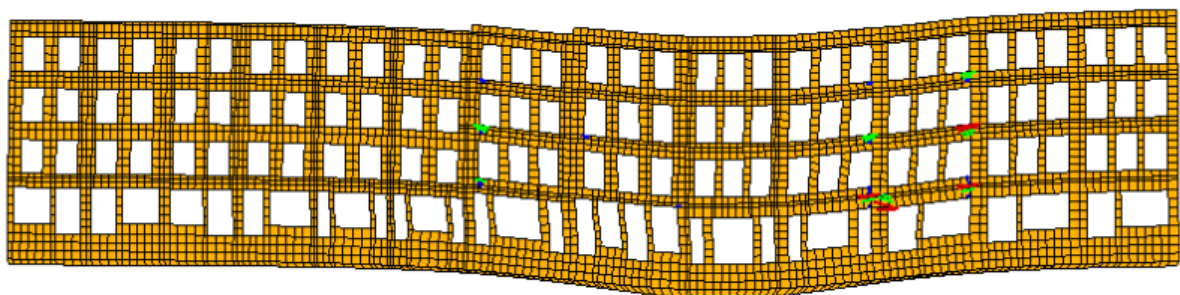
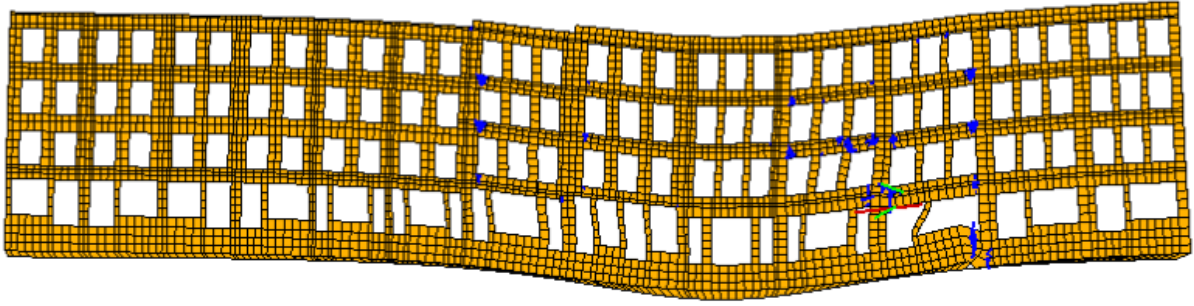
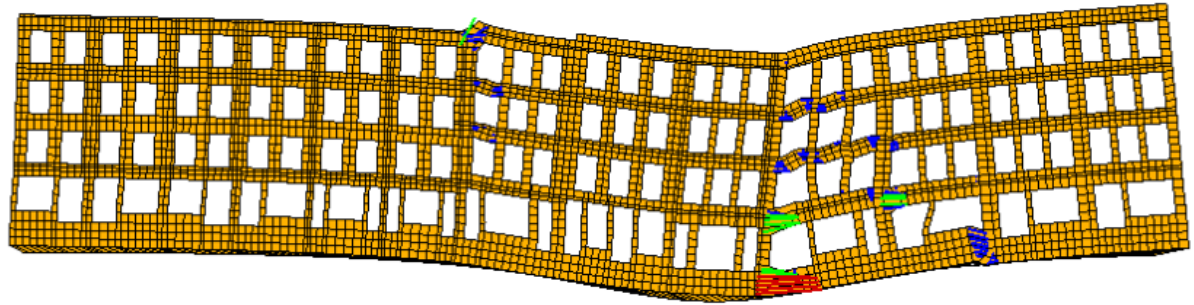


Figure 55: crack pattern at $V_L=1.1\%$

Figure 56: cracks pattern at $V_L=1.4\%$ Figure 57: structural damage at $V_L=1.8\%$

The figures show that the cracks are initiated in the corners of the windows and propagate to the other windows. The cracks are situated in the facades near the inflection point distance of the applied settlement trough, where the shear distortion is greatest and the curvature is lowest. The number and width of cracks grow up to $V_L=1.8\%$ where the major crack appears at the bottom of the façade. The initial cracks have closed; the big crack causes the other part of the facades to relax. In reality of course it is not possible that a crack closes totally; some damage will always remain.

The local behaviour of the facades introduces the cracks in the facades. In the linear model the strains were extracted from this global behaviour. In accordance the “beams” cracked in the nonlinear model are marked red. See Table 30.

V_L (%)	0.237		0.497		0.783		1.096		1.436		1.805	
Beam nr	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)	ϵ_b (%)	ϵ_d (%)
L1	0,007	0,007	0,01	0,009	0,013	0,01	0,017	0,013	0,018	0,016	0,019	0,018
L2	0,012	0,012	0,021	0,015	0,028	0,022	0,038	0,028	0,04	0,032	0,046	0,039
L3	0,017	0,016	0,032	0,017	0,043	0,03	0,051	0,038	0,065	0,041	0,074	0,05
L4	0,017	0,02	0,03	0,024	0,041	0,03	0,049	0,037	0,059	0,041	0,066	0,047
L5	0,005	0,015	0,008	0,014	0,015	0,012	0,021	0,012	0,020	0,012	0,029	0,012

Table 30: Bending and shear strains in the governing facade

The table shows that the first cracks appear at a tensile strain of 0.04%. The middle three beams crack as they reach this limit strain at the same time as the crack occurs in this beam. This can be seen in Figure 54 to Figure 57. The beam at the bottom (L5) should also show cracks at $V_L=1.4\%$, but the strains in the linear model have not reached the crack strain yet (0.02% instead of 0.04%). This can only be explained by the redistribution of forces through the model after cracking; the uncracked parts of the façade have to compensate for the cracked parts. This is shown in Figure 58 and Figure 59. The bending and shear strains are extracted at the place of the final crack for both the linear and nonlinear model.

The final crack can be said to be caused by the redistribution of the forces in the facades due to the cracked parts, causing the strains at the bottom part to increase.

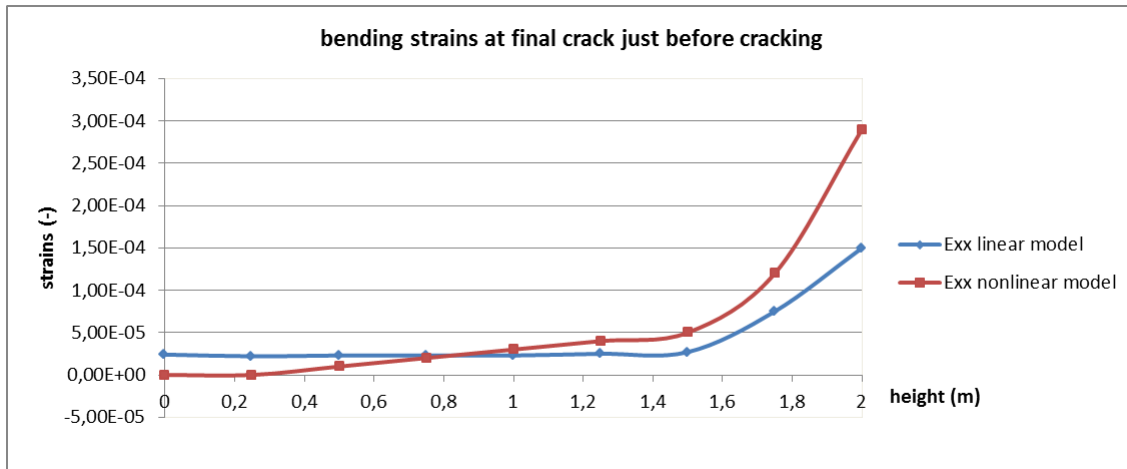


Figure 58: Bending strain in the linear and nonlinear model at $V_L=1.4\%$

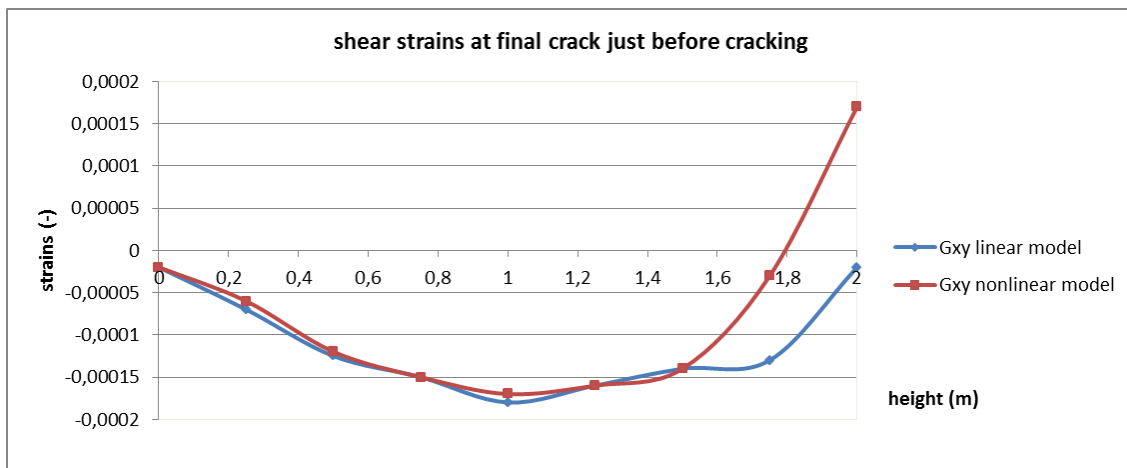


Figure 59: Shear strain in linear and nonlinear model at $V_L=1.4\%$

The figures show that at the nonlinear model combination of the bending and shear strain lead to the crack at $V_L=1.4\%$

The nonlinear effects, i.e. redistribution of forces through the model after cracking, can be considered to be important for the crack propagation.

6.2.2 Post cracking deformation

For the increased imposed settlements as well the curvature and the shear distortion are determined. It is interesting to obtain whether the global behaviour of the facades changes after the first crack is initiated, if curvature and shear distortion is considered. The results for $V_L=0.78\%$ (first cracks) and $V_L=1.4\%$ (just before failure) are given in the figures below for both the linear and nonlinear masonry model.

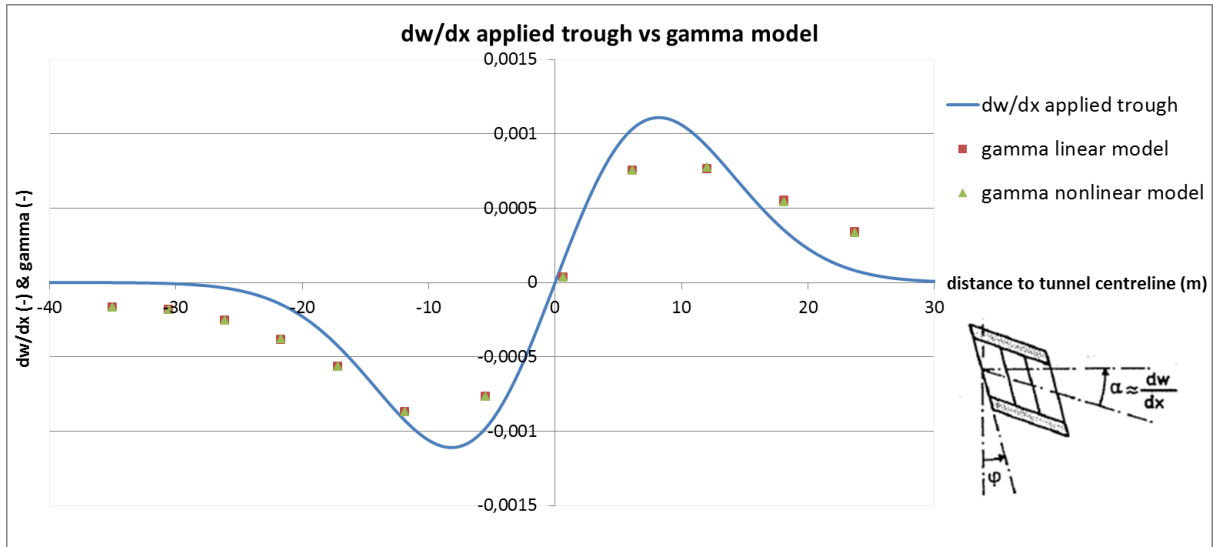


Figure 60: Shear distortion of the linear and nonlinear model at $V_L=0.78\%$ (appearance first cracks)

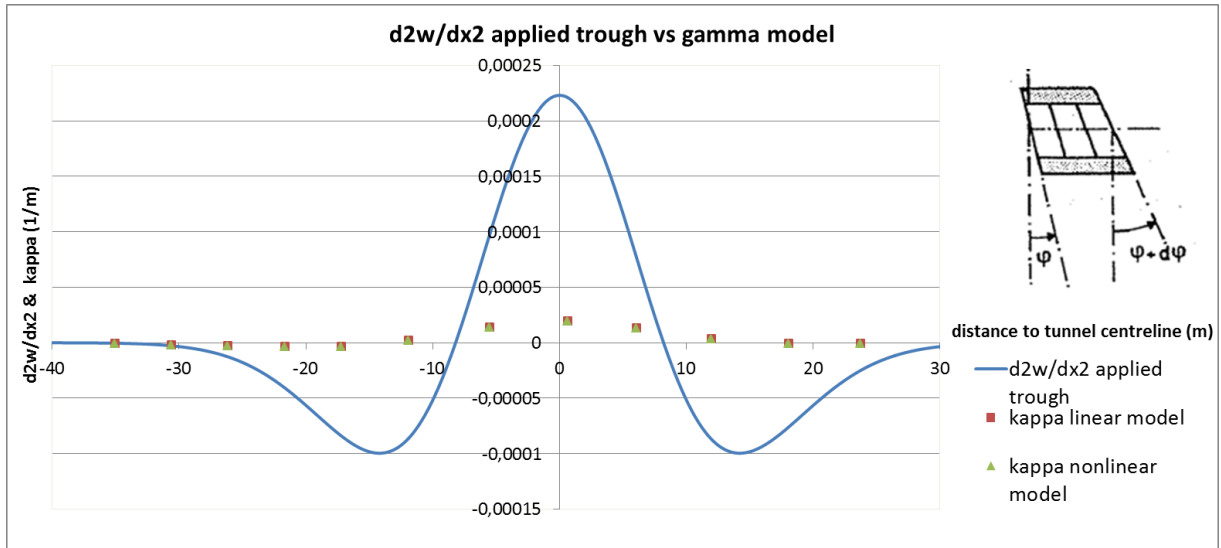


Figure 61: Curvature of the linear and nonlinear model at $V_L=0.78\%$ (appearance first cracks)

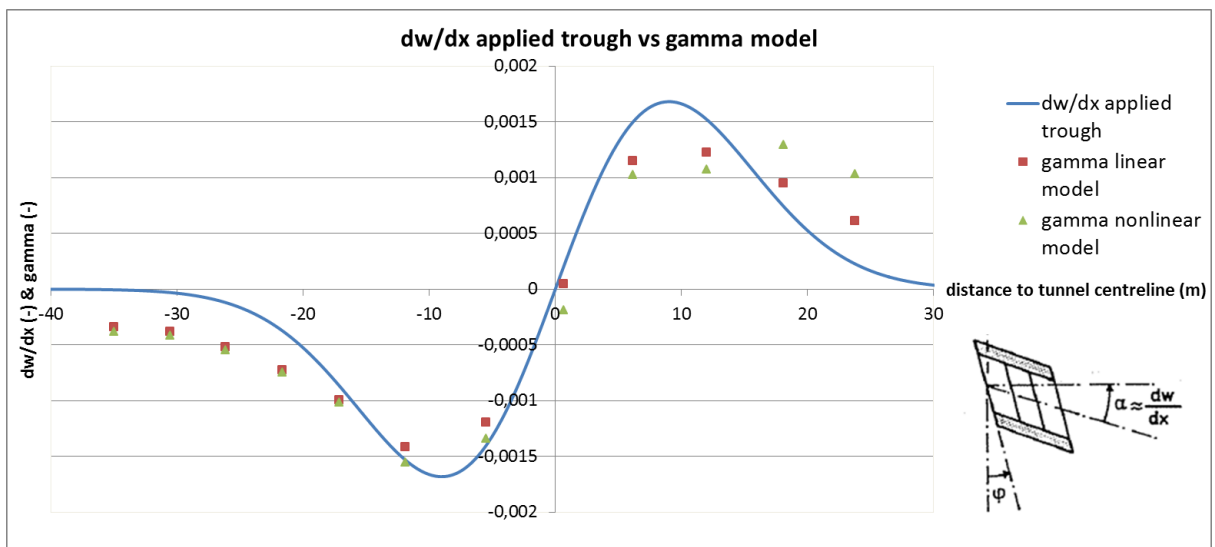


Figure 62: Shear distortion of the linear and nonlinear model at $V_L=1.4\%$ (just before structural damage)

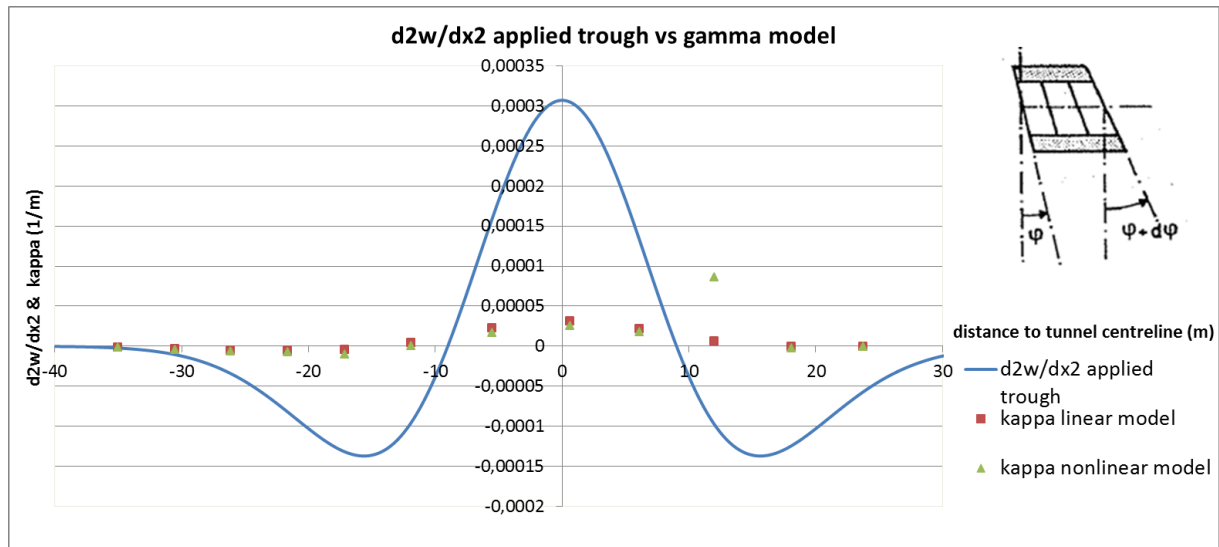


Figure 63: Curvature of the linear and nonlinear model at $V_L=1.4\%$ (just before structural damage)

The figures show that the overall behaviour of the facades in linear and nonlinear analyses remains the same; great shear distortion and low curvature of the facades. Only locally, at the position of the façade about to show structural damage, the curvature of façade has increased at $V_L=1.4\%$, just before cracking. This is a result of the presence of cracked parts; the cracked parts have lost their tensile capacity and the uncracked parts are activated. Since the uncracked parts are located at the top and bottom of the façade, bending deformation is induced. The façade shows bending action to compensate for the cracked parts.

It can be concluded that the global behaviour remains the same for the model at different volume loss ratio's; only at the position of the crack, just before appearance of the structural damage, bending action is present. The model is mainly subjected to shear distortion, even in the post-cracking stage.

6.3 Damage numerical model compared to damage predictions.

In Paragraph 6.2 the crack widths of the numerical model were determined with nonlinear analyses. The damage prediction methods examined in Chapter 5 will be considered again, but now for comparison with the nonlinear numerical model. The strains calculated with the damage prediction models can be translated to crack widths through Table 3: Bandwidths for limiting tensile strains related to damage (Bre (1981), (1990) and Burland et al. (1989) in Chapter 2. The predicted damage can be compared to the damage found in the numerical model. The reliability of damage prediction methods under the assumed tensile strength can be determined. It is emphasized that the assumed tensile strength has a great influence on the results, as is explained in Paragraph 6.1.

It should be emphasized that the crack band widths in Table 3 are based on ease of repair, not the actual crack width occurring in the façade or wall per se. These crack band widths shall be compared to the crack widths found in the numerical model, which will be assumed to be the actual crack cracks widths. As can be seen in Paragraph 6.2, the cracks up to a volume loss ratio of 1,4% (5 mm) are only aesthetical damage, but at $V_L=1.8\%$ a major crack occurs over which propagates through almost the entire height of the model. This can be considered to be structural damage; the part “functional damage” is skipped in the model. If a crack width of 15 mm is found with one of the methods, this will be qualified as structural damage.

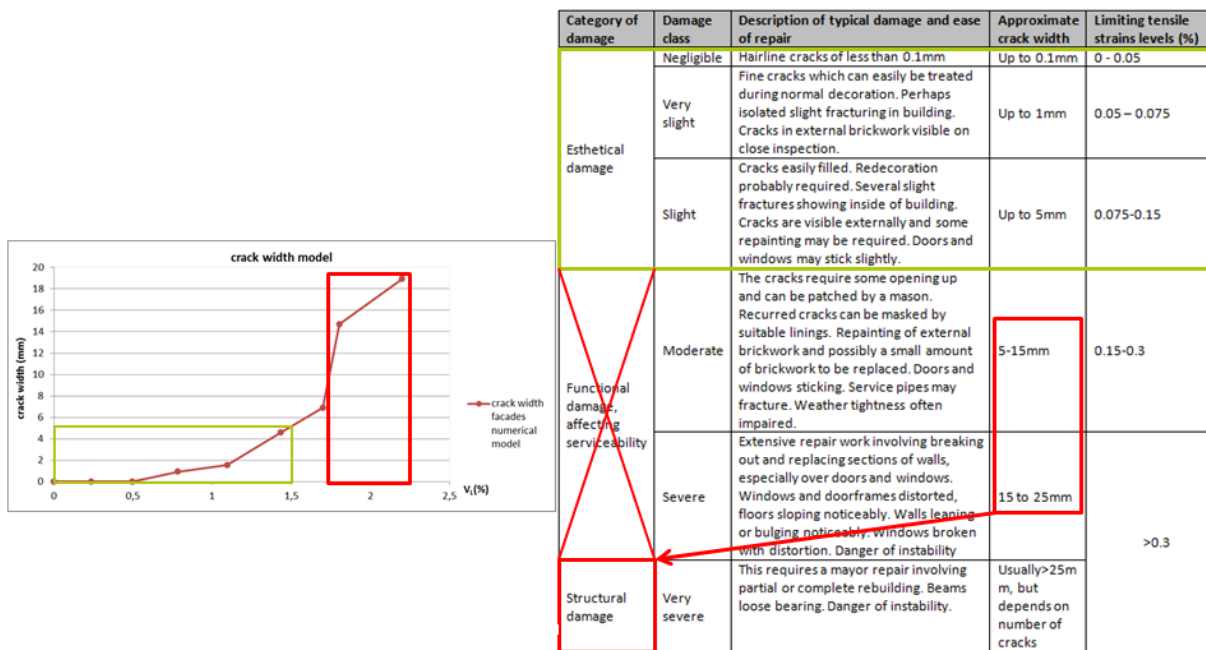


Figure 64: The crack width in the model skips the category “ functional damage”.

6.3.1 Comparison with LTSM with $E/G=2.6$

The strains calculated with the LTSM are translated to a damage classification through Table 3. The damage classification determined with the LTSM is compared with the damage found in the nonlinear numerical model. The upper and lower bound of Table 3 are both provide in the figure. The comparison can be found in Figure 65.

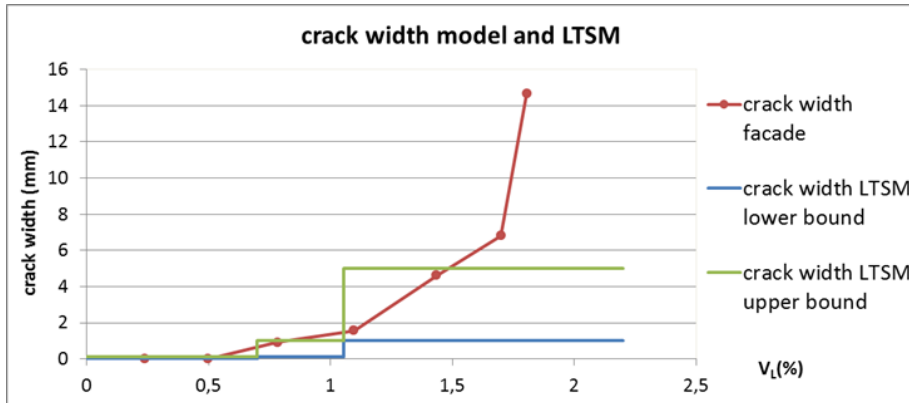


Figure 65: crack width in the numerical model compared to the crack width determined with the LTSM.

The figure shows that the LTSM gives a good indication of the damage to be expected up to a volume loss ratio of 1.4%, but underestimates the damage at higher volume loss ratios. This underestimation can be blamed on one of the simplifications of the LTSM; the LTSM does not take into account the nonlinear effects due to cracking.

It can therefore be concluded that the LTSM provides a reliable indication of damage of a volume loss ratio up to 1.4%.

6.3.2 Comparison with LTSM with $E/G=12.5$

The crack widths are again determined with the calculated strains and Table 3. The damage for the LTSM with $E/G=12.5$ is illustrated in Figure 66.

The figure shows that also the LTSM with $E/G=12.5$ only provides a reliable damage classification up to a volume loss ratio of 1.4%. Also this can be blamed on the nonlinear effects not taken into account in the LTSM.

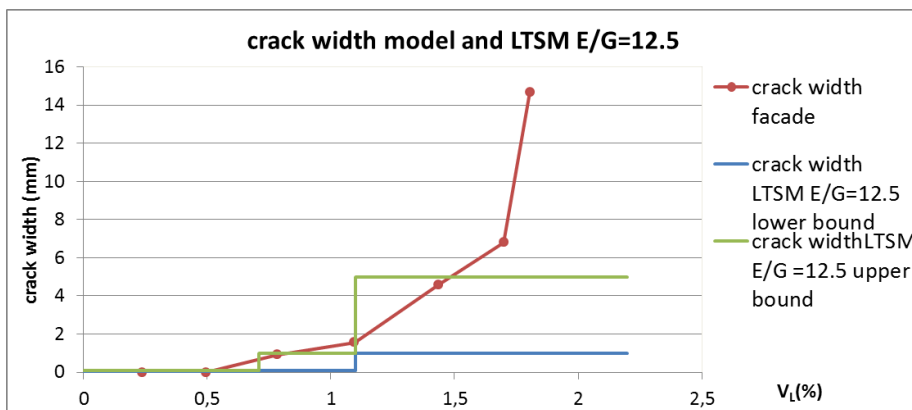


Figure 66: crack width of the numerical model and the LTSM with $E/G=12.5$

6.3.3 Comparison with portal frame method

The strains in the portal frame method are translated to damage classification of Table 3. The results are illustrated in Figure 67. The figure shows that in most cases the portal frame model underestimates the damage to be expected (Figure 67). It should be noted that the strains of the beams in the portal frame method are considered only; the largest bending strain is chosen. The damage classification only gives a reliable indication up to 0.78%.

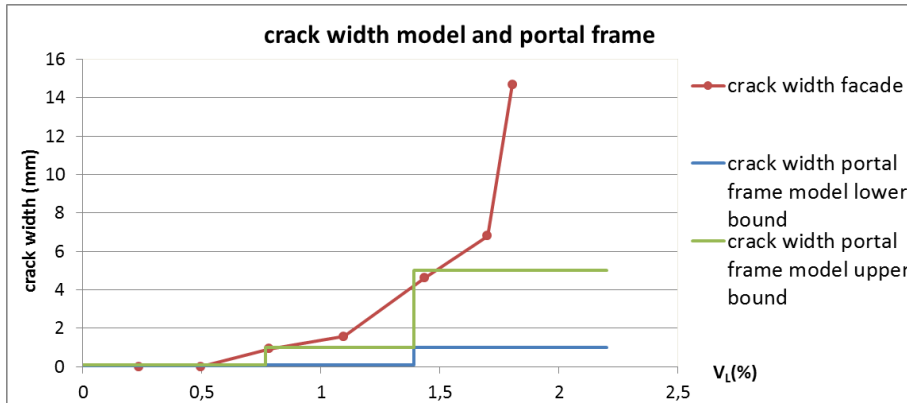


Figure 67: Crack widths for the portal frame model compared to the numerical results

6.3.4 Comparison with Forget-Me-Not model

The strains are used to compute the damage according to Table 3. The results are given in Figure 68.

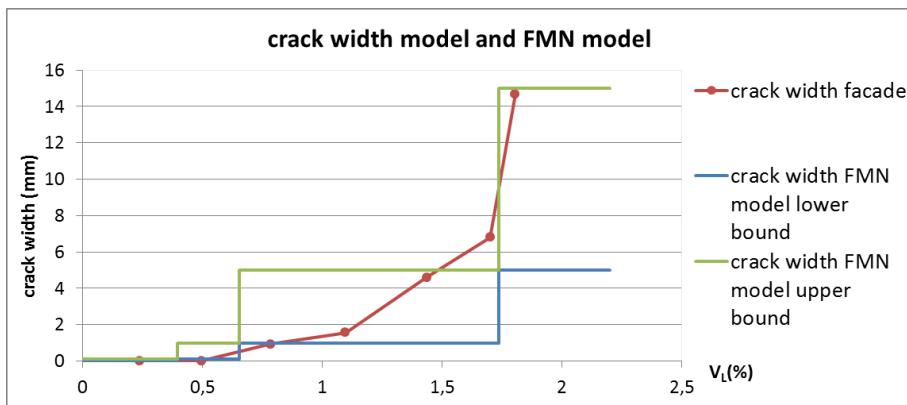


Figure 68: Crack widths of the FMN model compared to the results of the numerical model

The figure shows that the FMN model gives a good indication for the damage to be expected at volume losses up to 2% if only the crack width is considered. For the FMN method it can be said that the overestimation of the strains, as mentioned in Paragraph 5.3.3, account for the nonlinear effects. The FMN model is the most reliable method of the four,

6.4 Comparison between damage prediction methods

The damage prediction methods can be compared to each other on the basis of the nonlinear numerical model. The results are based on the assumption of the tensile strength of the masonry.

For the nonlinear model the deformations do not change; this was concluded in Paragraph 6.2. For comparison with the nonlinear numerical model it is not important that the strains predicted are exactly right, as long as the predicted damage is consistent with the damage obtained in the nonlinear analyses. The main goal of this chapter is to find a reliable damage prediction method. Therefore a comparison is made on the following points:

- Deformation consistent with global behaviour numerical model: the applied method implies that shear distortion is governing over curvature, as was obtained in the global behaviour of the numerical model.
- Deformation consistent with local behaviour numerical model: the applied method implies that individual parts of the façade are curved, despite the fact the façade globally acts in shear deformation.
- Strains consistent with global behaviour numerical model: the applied method implies that shear strain are governing over bending strains, as was obtained in the global behaviour of the numerical model. The magnitude of strains is neglected.
- Strains consistent with local behaviour numerical model: the applied method implies that bending strains are governing over shear strains due to bending of individual parts of the façade, as was obtained in the local behaviour of the numerical model. The magnitude of strains is neglected.
- Damage classification: the applied method gives a good indication of the damage to be expected up to a certain volume loss ratio

The results are listed in Table 31.

		LTSM E/G=2.6	LTSM E/G=12.5	Portal frame method	FMN model
Deformation	Consistent with global behaviour (<i>shear distortion over curvature</i>)	No	Yes	Yes (shear governing assumed in model)	Yes (shear governing assumed in model)
	Consistent with local behaviour (<i>act like portal frame/bending beams</i>)	No	Yes (with E/G=12.5 a portal frame is assumed)	Yes	Yes
strains	Consistent with global behaviour (<i>shear over bending</i>)	No	Yes	No	No
	Consistent with local behaviour (<i>bending over shear</i>)	Yes, but bending in global sense	No	Yes	Yes
	Damage classification	Up to 1.4%	Up to 1.4%	Up to 0.78%	Up to 2.0%

Table 31: comparison damage prediction models to local and global behaviour for nonlinear analyses

If only conclusion would be drawn upon the final results (i.e. damage prediction), the FMN model is the safest method to predict damage if compared with the nonlinear numerical model; it seems to take into account the nonlinear effects of the facades, as was mentioned before. But conclusions should be drawn upon all factors in Table 31.

For the classical LTSM ($E/G=2.6$) it can be stated that it is not consistent with the global behaviour of the numerical model for both strains and deformation; it can be concluded that the classical LTSM does not give a good representation of the global nor the local behaviour of the facades. The FMN model and the LTSM with $E/G=12.5$ performs better in both criteria. The classical LTSM is considered to be the worst method of the three methods left.

The FMN represents the failure mechanism of the numerical model (bending of parts of the façade that cracks due to shear distortion) while the LTSM with $E/G=12.5$ is consistent with the global behaviour of the facades, but is not consistent with the failure mechanism. Both models give a reliable indication of the damage to be expected at low volume loss ratio (up to 1.4% at least), but the FMN model is more conservative which is necessary considering the brittle crack in the numerical model at $V_L=1.8\%$. The LTSM with $E/G=12.5$ is unsafe if a steeper settlement trough is applied to the model; it will underestimate the damage. A great advantage of the FMN model is that not both sagging and hogging zone has to be evaluated; the first derivative of the settlement trough is enough to determine the expected damage in a façade, but the individual stiffness's of different parts of the façade should be known. Another disadvantage is that it is not known whether this method is in general applicable for facades, although this is also the case for the LTSM.

Take all the aforementioned arguments into account, it can be concluded that the FMN model is the best model to predict damage in the case of the nonlinear numerical model for the Daniel Stalpertstraat.

7. Evaluation results linear and nonlinear models

The conclusions from the linear and nonlinear analyses are different. For the linear analyses it was concluded that the LSTM with $E/G = 12.5$ is the best prediction method in case of the Daniel Stalpertstraat. According to the nonlinear analyses however, this method is not conservative enough; it underestimated the damage at high volume losses and the Forget-Me-Not method is safer. It is difficult to determine which of both the best method is for damage prediction. The numerical model is based on too many assumptions to draw conclusions upon this, certainly in case of the nonlinear numerical model. The main problem is that no damage was found under the imposed settlements; if damage did occur, model properties could be determined with more certainty. Especially the tensile strength is an important factor. This tensile strength especially influences the nonlinear model, providing a weak basis for the reliability of the nonlinear model and the conclusions drawn from this. The FMN method however is as the LSTM easy in use and sure has potential to provide better results than the LSTM.

General applicability

In order to draw a better conclusion about the method best suitable for the general case of facades subjected to transversal settlement trough, more case studies should be examined as well where both methods will be tested. The FMN model is a good model for the Daniel Stalpertstraat under the assumed tensile strength, but it might not be in other cases. On the other hand, the E/G ratio in the LTSM depends on the perforation ratio, as was stated by Cording et al. (2007). This is illustrated in Figure 69.

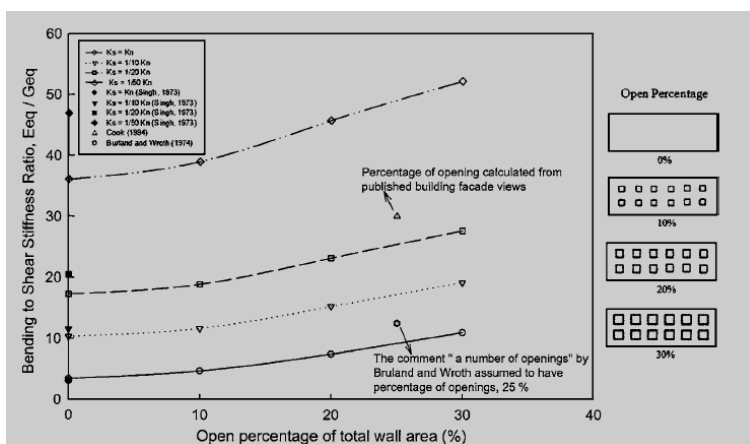


Figure 69: relationship between equivalent bending to shear stiffness ratio and open percentage of a wall, Cording et al (2007)

It would therefore be useful to examine the applicability of the FMN model to overcome E/G ratio depending on perforation rate. Future work could for instance determine a ratio for which the FMN model is applicable, and therefore the LTSM with a certain E/G ratio is no longer necessary.

Future work should provide more insight in the applicability of both methods under general cases of facades subjected to a transversal settlement trough.

8. Conclusions and recommendations

8.1 Conclusions

In this paragraph the main conclusions of the thesis are listed.

Case study: the case study of the Daniel Stalpertstraat was used to get insight the response of facades under a transversal settlement trough, induced by the tunnelling process. The results of the Daniel Stalpertstraat were compared to results of adjacent streets. The main conclusions are summarized.

- Soil surface settlements were measure to be max 3 mm; the vertical building movement were measured up to 4 mm. The tunnel induced settlements did not cause the facades to crack.
- The maximum vertical movement of the houses is in between the maximum settlement of the soil surface and first sand layer.
- In the comparison with other streets is was found that the building response is different for every row of houses, depending on foundation level, quality of the foundation and geometry of the houses

Numerical model: a numerical model was made with the finite element software program DIANA. Using the results of the case study the numerical model was calibrated to match the measurements of the case study. The main conclusions of this part are summarized below:

- A semi-coupled numerical 2D model of the facades is created subjected to a transversal settlement trough, which is tweaked to match the measurements of the case study. The applicable Young's modulus of the masonry is 1000 N/mm^2 and the normal interface stiffness is $1.2 \cdot 10^7 \text{ N/m}^3$.
- The deflection of the facades is caused mainly by shear distortion; curvature of the facades is low. The openings in the facades have significantly reduced the shear stiffness of the facades.
- The numerical model is sensitive to boundary conditions.

Linear analyses of increased settlements: The imposed settlements in the numerical model were increased up to a volume loss ratio of 2% to examine the behaviour at larger settlements. Linear analyses of the numerical model under the increased settlements were performed. A distinction is made between the local and global behaviour of the numerical model. The strain and deformations found in the linear numerical model were compared with strain and deformations found in 4 damage prediction methods. The main conclusions of this part are summarized in the list below:

- It was concluded that a distinction has to be made between local and global behaviour of the numerical model to explain the results of the numerical model. The global behaviour treats the behaviour of the facades as a whole, the local behaviour is the behaviour of an individual facade
- If the global behaviour of the facades is considered, the shear strains are larger than the bending strains. The shear strains however do not provide the maximum tensile strain in the numerical model; these are 30% lower than the maximum principal strains. The results of the LTSM with $E/G=2.6$ and LTSM with $E/G=12.5$ were examined as a damage prediction method for facades. The LTSM with $E/G=12.5$ shows good comparison with the global behaviour of the numerical model in terms of strains and deformation.
- The study of the local behaviour showed that an individual façade deforms like a portal frame under shear. This behaviour introduces local bending strains that provide the maximum tensile strain in the numerical model; shear deformation results in maximum

- bending strains. Two damage prediction models for the local behaviour, based on classical beam theory, were examined for their applicability; the Forget-Me-Not method and the portal frame method. Both models do not calculate the same strains as were found in the results of the analyses of the local behaviour of the numerical model.
- The LTSM with $E/G=12.5$ was found to provide the best results in terms of strains and deformation if compared with the linear numerical model.

Nonlinear analyses of imposed settlements. Nonlinear analyses of the increased settlements were performed, under assumptions of the tensile strain. The post cracking behaviour of the numerical model was examined. The crack widths of the nonlinear numerical model are compared to the results of the crack width found in the 4 damage prediction models used in the linear analyses. The crack widths in the damage prediction models were obtained by translating the strains in the damage prediction model to crack widths. In this way the safety (conservativeness) of each model can be evaluated. The main conclusions of this chapter are:

- The behaviour of the facades does not change because of cracking; large shear distortion and low curvature of the facades remain. The ratio between shear distortion and curvature remains 40.
- At volume loss ratio of 0.78% the first cracks appear in the corners of openings in the façade. The cracks occur at the same distance as the inflection point distance of the applied settlement trough, where the shear distortion is greatest. Cracks gradually increase up to a volume loss ratio of 1.8%, at which a major crack occurs, propagation over the height of the façade.
- The Forget-Me-Not method is the safest model if compared with the nonlinear numerical results. It provides good comparison with the nonlinear numerical model under the assumed tensile strength at large settlements.

8.2 Recommendations

In this paragraph the recommendations are summed that follow from the thesis:

- In future tunnelling projects it would be useful to measure both the horizontal and vertical movements of the soil over the height of the soil. This can provide a better insight in the development of soil movements over the height of the model, which can be used to optimise numerical soil settlement prediction models. If this is done in parts both near buildings and under greenfield conditions, more insight can be gained in soil-structure interaction.
- Prisms attached to the bottom of the houses can help to determine the transference of horizontal soil movements to the houses.
- The numerical model parameters obtained are based on the assumption of tensile strength and fracture energy of the masonry in the Daniel Stalpertstraat. A verification of the properties of the masonry in this street would help to improve the reliability of the numerical model, for instance by testing pieces of old masonry.
- The numerical model does not take into account the 3D effects of the houses; these could influence the building response. A verification of the 3D effects on the building response can determine the significance of the 3D effects.
- The applicability of the forget-me-not model and the adapted LTSM should be evaluated by more case studies of facades subjected to a transversal settlement trough. It could be possible that for instance a reduction factor for allowable tensile strains, based on the amount of perforation of a façade, could improve the adapted LTSM method. The case studies should provide more insight in the general applicability of the Forget-Me-Not model for facades.

9. Literature overview

Abeelen, R.A.F. van (2009). *Modelling of building response to imposed settlements, a critical review of current modelling techniques*.

Addenbrook, T, and D. Potts (1998). The prediction of building performance in response to tunnelling. In *Prediction and performance in Geotechnical Engineering*, pp. 357-392.

Bloodworth, A. (2002). *Three-dimensional analysis of tunnelling effects on structures to develop design methods*. PhD thesis, University of Oxford.

Boscarding, M., and E. Cording (1989). Building response to excavation induced settlements. *Journal of Geotechnical engineering*, pp. 1-21

Bouma, A. (1989) *Mechanica van constructies, Elastostatica van slanke structuren*. Delft University Press

Burland, J.B, and J.R. Wroth (1974). Settlement on buildings and associated damage. In *Conference of on settlment of structures*, Cambridge, England, pp. 611-654.

Burland, J.B, J.R. standing and F.M. Jardine (2001). *Building response to tunnelling, case studies from construction of the Jubilee line extension, London*, pp. 23-43.

Burland, J.B, F. Jardin and J. Standing (Eds) (2001). *Building response to tunnelling*. CIRIA Franzius, J. (2003). *Behaviour of buildings due to tunnel induced subsidence*. PhD. Thesis, University of London.

Giardina, G., M. Hendriks and J. Rots (2010). *Limiting Tensile Strain Method: voor tunnelbouw en diepe ontgravingen*. Delft University of Technology

Giardina, G., M. Hendriks and J. Rots (2009). *Assessment of the settlement vulnerability of masonry building*.

Giardina, G., M. Hendriks and J. Rots (2008). *Numerical analyses of tunnel-induced settlement damage to a masonry wall*. Delft University of Technology

Giardina, G., M. Hendriks and J. Rots (2009). *Coupled analyses of building damage due to tunnelling*. Delft University of Technology

Hartsuiker, C (1999). *Toegepaste Mechanica, Deel 1: evenwicht*. SDU uitgevers.

Hendriks, M. and J. Rots (1995). Onderzoek naar te verwachten zettingsschade bij de aanleg van de Noord/Zuidlijn in Amsterdam, diana studies fase2. Technical report 95-CON-R0343. TNO Bouw.

Kuijten, R.S., and G.J. van Doornik (2010). *CUR 532 Gebouwenresponsie*. Inventarisatie bestaande definities en predicties geotechnische gebouwschade. Grontmij Nederland bv.

Netzel, H. (2009). *Building response due to ground movements*. PhD Thesis, Delft University of Technology.

- Oversteegen, M.J. (1992). *Onderzoek kwaliteit fundering & casco, basisrapport*. Bouw- en Woningdienst Amsterdam.
- O'Reilly, M.P., and B.M. New (1982). *Settlements above tunnels in United Kingdom – their magnitude and prediction*. Tunnelling 82, M.J. Jones (Hrsg.), S. 173ff., IMM London.
- Peck, R.B (1969). Deep excavations and tunnelling in soft soil, Proc. 7th Int. Conference on Soil Mechanics and Foundation Engineering, pp. 225-290
- Son, M., and E.J. Cording (2005). Estimation of Building Damage to Excavation Induced Ground Movements. In *Journal of Geotechnical and Geoenvironmental Engineering ASCE*.
- Son, M., and E.J. Cording (2007). Evaluation of Building Stiffness for Building Response Analysis to Excavation Induced Ground Movements. In *Journal of Geotechnical and Geoenvironmental Engineering ASCE*.
- Sluys, L. (1999) *Computational Methods in Nonlinear Solid mechanics, Lecture Notes CT5142*. Delft Univeristy of Technology.
- Timoshenko, S. and S.J. Gere (1972). *Mechanics of materials*. Littion Educational Publishing.
- TNO DIANA bv (2005), *DIANA's user manual* (release 9.4 ed.). TNO DIANA bv.
- Tol, A. (2003) *Foundation Engineering and Underground, Lecture Notes CT5330*. Delft University of Technology.
- Vent, I.A.E. de, J.G. Rots and G.J. Hobbelman (2009). *Finite elements analyses help develop a diagnostic aid for masonry*. Delft University of Technology
- Wells, G. (2006). *The finite element method: an Introduction, Lecture Notes CT5123*. Delft university of technology.