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NONLINEAR NUMERICAL SIMULATION OF SETTLEMENT-INDUCED DAMAGE TO SOLID MASONRY WALLS

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Abstract

This paper studies the soil-structure interaction effects on the prediction of settlement induced damage in structures of cement-based material with nonlinear numerical FE-calculations. The empirical, analytical limiting tensile strain (LTS) method is used commonly in engineering practice to predict building damage due tot excavation induced ground deformations. The LTS gives in general a conservative approach of the expected damage on the buildings as the soil-structure interaction is neglected.
This paper discusses a more advanced, numerical approach to predict settlement building damage and to study the effects of soil-structure interaction on the damage results on masonry bearing walls. Various soil to structure stiffness ratios are considered, ranging from stiff to soft soil. The study is focused on solid masonry walls, positioned symmetrically over the sagging zone of a green field settlement trough. The limited tensile and compressive resistance of masonry is considered via a Rankine-Hill material law. The considerations in this paper are restricted to the effects of vertical ground movements only (horizontal ground movements are not within the scope of this paper).

Key Words

Settlement, Masonry, Finite elements, Soil-structure interaction, Cracking

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1 Introduction

Empirical analytical prediction methods are currently used for the prediction and classification of settlement-induced damage in structures of cement-based material (Netzel 2004). The LTS (limiting tensile strain method) is used to predict strains in adjacent buildings due to excavation induced ground deformations. Figure 1 shows the principle of the method for a building in the sagging zone and the influence of vertical settlements. It should be noted that the developers of the method (Burland & Wroth 1974 and Boscardin & Cording, 1989) have chosen a fictive central point load to fit the deflection profile caused by tunnelling. The influence of a fictive uniformly distributed load was investigated (Burland & Wroth (1974)) and it was concluded, that the relationships for the critical strains are not sensitive to the type of loading.

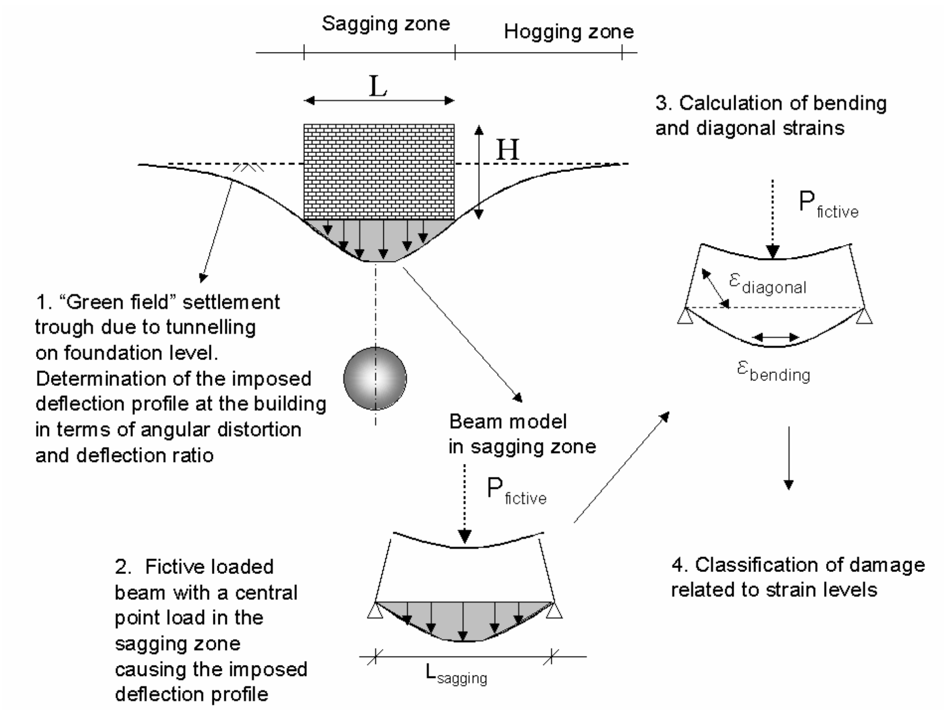


Figure 1: Principle of the limiting tensile strain method for the influence of vertical differential settlements in the sagging zone.

This paper will present the results of a numerical approach for the damage prediction in solid masonry walls, subjected to tunnelling-induced settlement with advanced non-linear FE-calculations taking into account the vertical soil-structure interaction and an advanced crack model for masonry. The finite element studies are refined to capture stress redistribution upon reaching and exceeding actual stress limits, both in tension and in compression in such settling walls. Cracking and crushing are considered via nonlinear computational material laws (Lourenço et al. 1998). A Rankine-Hill principle stress limit function is employed, with inclusion of post-limit strain-softening stress degradation. By this analysis, the simplifying assumption that large elastic strains relate directly to wide cracks in brittle cement-based material, is tested.

The difference between the greenfield deflection ratio (mid-span deflection Δ to span length **L,** see figure 1) and the deflection ratio as experienced by the interaction of the

solid masonry wall with the soil ($\Delta/L_{interaction}$) is quantified. For this study, solid masonry walls, positioned symmetrically over the sagging zone of a green field settlement trough, are considered.

2 Soil to wall interaction

Figure 2 shows schematically the phenomenon of the soil to wall interaction due to imposed ground deformations on a masonry bearing wall. Notice how the edges of the wall actually undergo more settlement than according to the green field trough profile. The deflection in the mid of the span is however reduced due to the soil-structure interaction. The deflection ratio is a criterion for differential settlements and used in the LTS to determine the distortion and the potential damage of a building.
A gap between the soil and the wall can develop at mid-span if both the bending stiffness and the yielding tensile strain ε_{yield} of the masonry wall are high enough in order for the wall to bridge the gap.

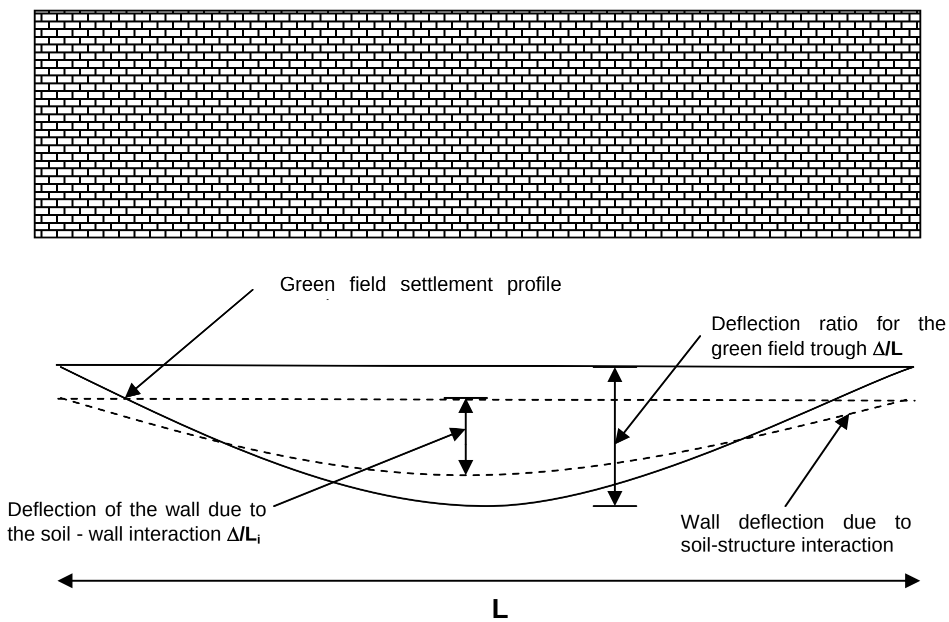


Figure 2 Effect of the soil to wall interaction on the Δ/L ratios.

A solid masonry wall carries floor loads which are transferred down through the wall and finally into the soil. These floor loads, together with the self-weight of the masonry, exert a pressure on the soil. The bedding (in terms of soil pressures) of the wall into the soil depends on the size of the wall, the soil properties and the width of the foundation. Due to soil-structure interaction the initial uniform loading of the soil due to the wall loads is redistributed if the ground deformations are imposed. This will lead to stress and strain concentrations in the wall, which can lead to cracking if the tensile resistance of masonry is reached. These effects are included in the calculations as the floor loads and the dead weight are applied.

3 Numerical calculations

3.1 Material parameters

3.1.1 Linear elastic

The linear elastic material parameters varied in this study are given in Table 1. For a parametric study, the stiffness of the soil E_{soil} is varied between practical ranges (see Table 1).

Table 1 Linear elastic masonry and soil material parameters used in this study.

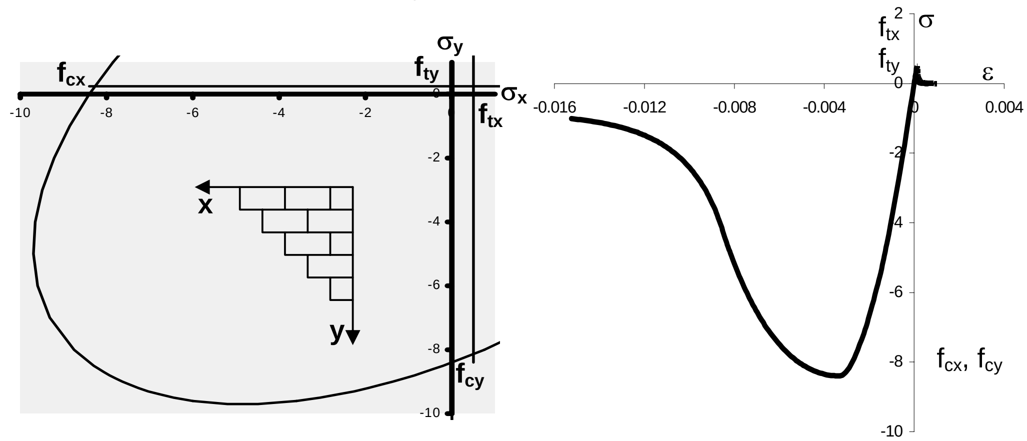
Masonry			
Parameter		Value	Unit
Young's modulus	E	4000	N/mm^2
Shear modulus	G	1667	N/mm^2
Density	ρ	1800	kg/m^3
Soil			
Parameter		Value	Unit
Young's modulus	E_1	10	N/mm^2
	E_2	50	N/mm^2
	E_3	80	N/mm^2

3.1.2 Nonlinear

The non-linear material properties for the interface between soil and the wall and the crack model for masonry are given in chapter 3.3.2.

The effect of stress re-distribution, crack initiation and propagation was considered by activating a nonlinear material law for the masonry. An anisotropic Rankine-Hill model (Lourenço et al. 1998), which captures the different tensile strength parallel to bed joints (f_{tx}) from the low tensile strength perpendicular to be joints (f_{ty}), was used, Figure 3. For lack of information, equal compressive strength in the two orthogonal directions was prescribed (f_{cx}, f_{cy}).

The strength reduction after the respective strains corresponding to the peak stresses in both tension and compression, as well as in both material directions, is governed by fracture energies $\mathbf{G_{tx}}$ and $\mathbf{G_{ty}}$ respectively for the two orthogonal directions in tension, and in compression by $\mathbf{G_{cx}}$ and $\mathbf{G_{cy}}$. See Table 2 for the values of these parameters.



(a) *(b)*

Figure 3 (a) Rankine-Hill strength limit function. (b) Stress-strain laws.

Table 2 Material parameters for the masonry nonlinear behaviour.

f_{tx} (MPa)	f_{ty} (MPa)	f_{cx} (MPa)	f_{cy} (MPa)	G_{tx} (N/mm)	G_{ty} (N/mm)	G_{cx} (N/mm)	G_{cy} (N/mm)
0.5	0.25	8.4	8.4	0.03	0.01	20	20

3.2 Loads

The walls are loaded under the influence of three different actions: (i) self-weight, (ii) floor loads at the levels of the respective floors and (iii) green field settlement of the soil calculated from a tunnel-induced settlement trough. The finite element program automatically generates the self-weight of the masonry walls from the modelled geometry and prescribed density. The applied floor line loads consist of the dead weight for timber floors in old masonry structures and 50% of the mobile loads. Taking a general span width of the floors of 7m a line load for each floor level of 11 kN/m for a common massive bearing wall in a block of masonry houses is applied.
The profile of the imposed soil deformations is derived by the analytical, empirical approach for TBM-tunnelling settlement profiles in soft soil (Netzel, 2004). The settlement trough in the sagging zone is described with a Gaussian curve using the input parameters as given in Table 3.

Table 3 Green Field settlement parameters employed throughout this study.

Parameter		Value	Unit
Depth of the tunnels axis	$\mathbf{z_o}$	22000	mm
Diameter of the tunnel	**D**	9500	mm
Point of inflection of the settlement trough	**I**	9900	mm
Trough width parameter	**K**	0.45	
Volume loss	$\mathbf{V_l}$	0.03 (3%)	

It should be emphasized that 3% volume loss for TBM–tunnelling is a very bad TBM-performance, which is an upper bound for practical ranges of volume losses. Nevertheless this value is chosen for this parametric study in order to observe significant effects (of stresses, strains and cracking) in the FE-calculations.

3.3 Modelling

3.3.1 Geometry

Figure 4 shows the general geometry, dimensions and configurations of loads for the FE-models. The number of floors supported by a wall depends on the height of the wall H. A general height of 3m between floors is used in this study. The first floor is situated at 0,5m above the bottom edge of the wall. A widened masonry foundation footing is included at the bottom of the wall over a height of 0,5m. The finite element program suppresses the initial displacements resulting from the first calculation stage (initial loading), but stores the stresses and strains in its memory for further development as induced by the prescribed displacements.

3.3.2 Interface between soil and structure

The soil is modelled by the interface with its stiffness properties. The interface has the stiffness characteristics of the soil for vertical stresses and is free for translation in horizontal direction (horizontal soil-structure interaction is not considered).

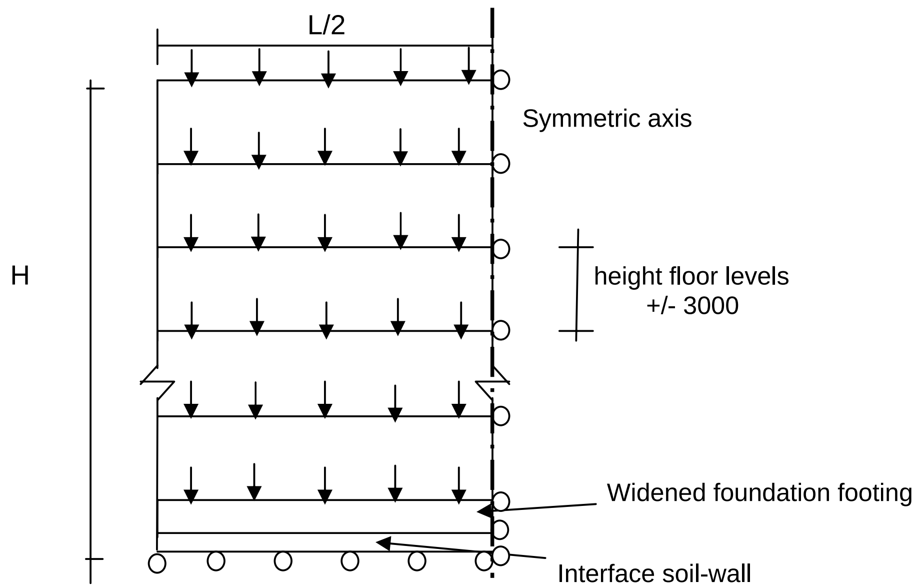


Figure 4 Schematisation of the solid masonry wall with soil interaction.

A nonlinear interface (Van Zijl 2000) is used to model the soil to wall interaction, without actually modelling the soil itself. The parameters of the interface relate the normal stresses σ_n at the integration points of an interface element to the vertical displacement δ_v between the corresponding upper and lower nodes of the interface element, according to Figure 5.

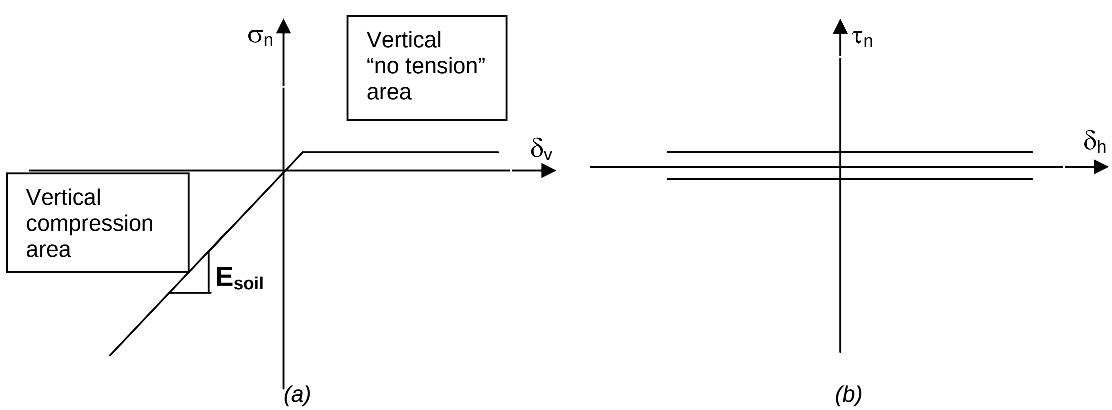


Figure 5 Relationship between interface (a) normal stress (σ_n) and vertical/normal opening displacement (δ_v), (b) shear stress (τ_n) and horizontal/slipping displacement (δ_v).

An insignificantly small shear limit is assigned to model free horizontal slip between soil and foundation at their interface. The walls under consideration in this study experience no soil to wall friction or lateral confinement caused by the horizontal soil-structure interaction. The properties of the horizontal interface are adapted to model this situation; Figure 5(b) defines the relationship between the absolute value of the shear stress τ_n developed at the integration points of an interface element and the horizontal displacement δ_h between the corresponding upper and lower nodes of the interface element.

The interface has nonlinear "no tension" properties, which means that only vertical compression stresses can be transmitted from the building to the soil, see Figure 5(a). It should be emphasized that the absolute vertical soil pressures are not restricted but they are checked manually, to guarantee that they do not reach unacceptable levels. A low tensile limit is prescribed to allow free vertical separation (no tension) between the foundation and the soil.

The prescribed displacements for the sagging zone are imposed to the interface element's bottom nodes.

3.3.3 Considered cases

Several cases were analysed including L/H-ratios in the range $0.5 \leq \mathbf{L/H} \leq 3$, two different wall lengths (19,8m and 9,9m) and the variation of the soil stiffness as given in Table 1.

3.4 Results of the FE-calculations

3.4.1 Interaction effect on the deflection ratio

Figure 6 shows the relation of the imposed greenfield ground deflection ratio $(\Delta/L)_{greenfield}$ versus the resulting wall deflection ratio $(\Delta/L)_{interaction}$ dependant on the L/H-ratio's and the length of the considered building. It should be noted that Figure 6 shows the results of the calculations with a non-linear ("no tension") interface-behaviour and the linear elastic behaviour of the masonry.

The diagram in Figure 6 shows, that the wall does not follow the greenfield soil settlement, confirming the schematised interaction effects of Figure 1. A considerable reduction of the deflection profile is caused by the soil–structure interaction. The soil-structure interaction effect is stronger (for the same building dimensions) with decreasing soil stiffness; in other words the imposed greenfield trough is more flattened at the building if the soil stiffness decreases. The influence of the structure length is also shown in Figure 6. The shorter wall (L=9,9m) shows a stronger interaction effect, thus more reduction of the imposed ground greenfield deformation for the same soil stiffness.

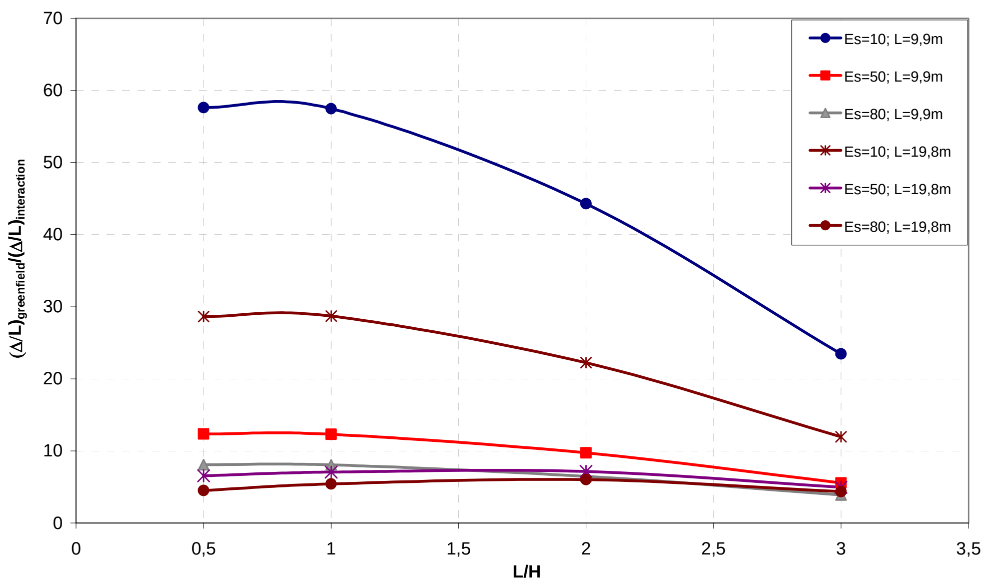


Figure 6 Interaction effect on the deflection ratio

Figure 7 shows the detailed development of the interaction effect for the building with the length of 19,8m depending on the absolute magnitude of the greenfield deflection ratio ($(\Delta/L)_{greenfield}$). An increase of the relationship $(\Delta/L)_{greenfield}$ / $(\Delta/L)_{interaction}$ is recognized with increasing absolute greenfield deflection ratio for the cases $E_s \geq 80$ MN/m^2 and L/H $\geq$ 1. This effect is related to the non-linear interface behaviour. The average value for ($(\Delta/L)_{greenfield}$ / $(\Delta/L)_{interaction}$) in the considered range of imposed greenfield deflection ratio is used for the presentation of the results in Figure 6.

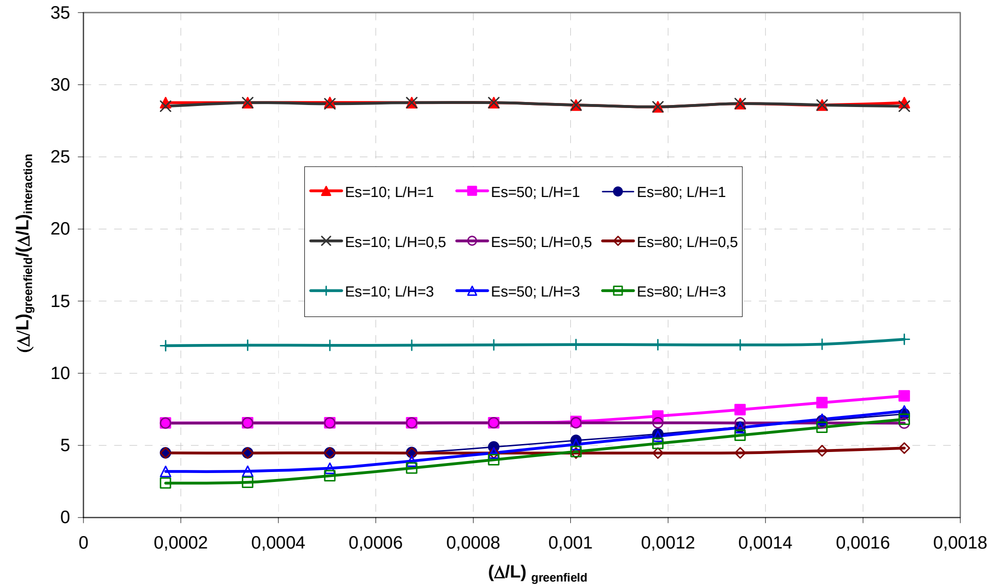


Figure 7 Development of the interaction effects dependant of the absolute magnitude of the greenfield deflection ratio for L=19,8 m

3.5 Linear versus non-linear results

The strain vectors in figure 8a show the redistribution of strains due to the imposed settlements for one example of the calculations (**L/H** = 2, **L** = 19.8m, $\mathbf{E_{soil}}$ = 80 N/mm^2). These calculation results include the non-linear interface behaviour and the non-linear masonry behaviour (smeared crack model).

The initially vertical loaded wall undergoes bending and shearing due to the imposed settlements. In the centre of the wall (at the symmetry axis) a gap develops between the wall and the soil, as the no-tension property of the interface is activated. The wall has to bridge this gap, leading to bending and diagonal strains in the wall and increased foundation pressure (Figure 8(e)) towards the outer ends of the wall.

A comparison between Figure 8(b) en Figure 8(c) shows the rapid increase of the tensile strains at the centre of the beam as soon as the tensile strength is reached at the bottom of the wall and cracking is initiated. Due to the brittle behaviour of the masonry the crack is propagating rapidly over the height of the wall.

A comparison between the results of the interaction relations for the linear masonry model and the non-linear masonry model is shown in Figure 9. It is shown that the linear approach is only valid up to a certain limit value. As soon as cracking develops at the bottom of the wall, the wall becomes significantly less stiff and will consequently almost follow the green field deflection ratio (decrease of the interaction factor to 1, which means no interaction reduction of the deflection ratio). Increasing ground distortions beyond the limit value will concentrate in the existing cracks, leading to an

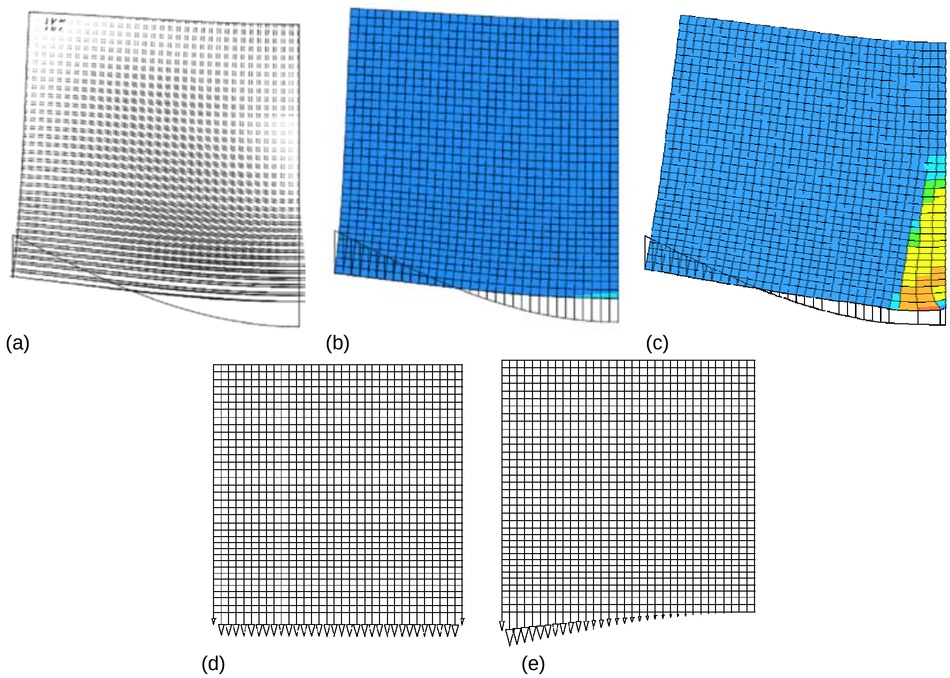


Figure 8 Computational results of the symmetric masonry wall response due to imposed sagging settlements just before and during cracking initiation and propagation for L/H = 2, L = 19.8m, E_{soil} = 80 N/mm^2. (a) Maximum principal strain vectors just before crack initiation. (b) Maximum principal strain contours just before cracking and (c) instantly after cracking initiates. (d) Resulting foundation support forces on the soil exerted by the wall under self weight and service loads (initial) and (e) after imposing the settlements.

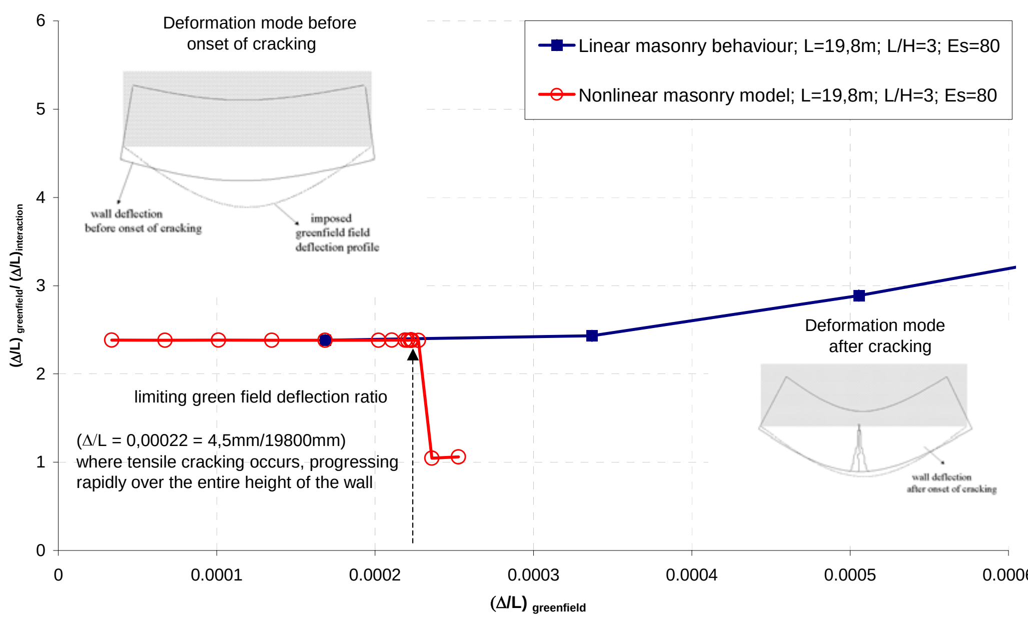


Figure 9: (($\Delta/L_{greenfield}$) / ($\Delta/L_{interaction}$)) for both linear and nonlinear masonry wall response for L/H = 3, L = 19.8 m and E_{soil} = 80 MN/m^2.

almost rigid body rotation of two separated wall parts accompanied with a rapid growth of the existing cracks (see deformation modes after and before cracking in Figure 8).

4 Conclusions

This paper describes a study of soil-structure interaction effects on the response of a massive masonry bearing wall due to imposed (excavation induced) ground deformations in a sagging mode. Different configurations for soil/wall-stiffness are investigated with advanced linear and non-linear numerical models. It is shown that up to a certain limit value for the imposed ground greenfield deflection ratio, the soil-structure interaction leads to a significant reduction of the greenfield deflection ratio, thus to a smaller and less damage causing distortion of the building. After the limit value is reached, cracking is initiated in the masonry. Cracking rapidly progresses over the entire height of the wall, which leads to an almost rigid body rotation of the cracked wall parts for increasing greenfield distortions beyond the limit value. The interaction effects beyond the limit value are almost negligible and the initiated cracks grow rapidly, because further imposed ground distortions are concentrating in the existing crack. The limit value is dependant on the combination of the building length, the L/H-ratio of the building and the soil stiffness and has to be determined with numerical non-linear calculations. It should be noted that this study is restricted to the influence of vertical ground settlements in the sagging mode and the assumption of a laterally unconfined wall.

5 ACKNOWLEDGMENTS

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