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RESTRAINED SHRINKAGE OF MASSIVE MASONRY WALLS

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Abstract

Restrained shrinkage of massive, one-storey, non-load bearing masonry walls with one-sided restraint has been simulated. A discontinuum model, including tensile and shear failure of mortar joints is employed to reproduce masonry behaviour in crack zones. Where cracking is not expected, masonry is modelled as an elastic continuum. The model is implemented in the commercial finite element programme ANSYS. The simulations show that cracking of walls is delayed if slip joints with low cohesion are applied at the wall foundation boundary or if weaker, lime rich mortars are employed instead of strong, cement rich mortars.

Key Words

Masonry, restrained shrinkage, cracking, simulation.

1 Introduction

If a massive wall is subjected to shrinkage while the foundation is left unaffected, free contraction of the wall is obstructed. This gives rise to tensile stresses in the wall and compressive stresses in the foundation. As the tensile strength of masonry is relatively low, the wall cracks. The largest tensile stresses occur at the boundaries between the wall and foundation at the ends of the wall and in a vertical section close to the middle of the wall.

Analytical methods developed by Copeland (1957) and Schubert & Glitza (1983) identify the length to height ratio of the wall, the tensile strength of masonry and the degree of restraint between the wall and foundation as the main parameters influencing the crack-free length of a wall subjected to restrained shrinkage. Although similar magnitudes of the crack free length are obtained, the employment of the finite element method in Lourenco (1996), CUR (1997) and van Zijl (1999) enables also the estimation of the effects of slip joints, different material combinations, loads and resulting crack width. The numerical modelling in Stehr (2002) indicates that the spacing of vertical movement joints to avoid cracks caused by restrained shrinkage can for brick masonry be increased by 10-30 % compared to the values obtained by analytical methods currently employed in Germany.

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In Sweden the recommended distance between movement joints is 12 -16 m for free-standing massive clay brick facades and 8 -12 m for facades consisting of calcium silicate masonry Mur90 (1990). Further, placement of a combined slip/water-proof joint between foundation and masonry wall is recommended. Nevertheless, no guidance is supplied concerning the mechanical properties of units, mortars or slip joint materials. In the present work, steps are made towards a better understanding of the behaviour of massive walls subjected to restrained shrinkage and a quantitative description of the main influencing parameters.

2 Modelling strategy

2.1 Geometry and boundary conditions

The simulations are carried out on a masonry wall resting on a concrete foundation, which restrains the wall from shrinking freely. As a reference case a 12 m long and 2.4 m high wall is considered. The height of the foundation is 300 mm, its width identical to that of the wall's, i.e. 120 mm. At the bottom, the foundation is fixed to the ground. To simulate a worst case scenario, rigid coupling between the wall and foundation is considered in the reference case. See Fig. 1a for details about the wall geometry, boundary conditions and the chosen modelling strategy.

2.2 Modelling details

In order to get a model of manageable size, only the central part of the wall is modelled in detail. The width of the area modelled in detail is 150 mm, corresponding to the length of a half brick and two head joints, i.e. 150 mm. In this area, bricks and mortar joints are modelled as separate materials, with a potential crack following the brick mortar boundary, see Fig. 1b.

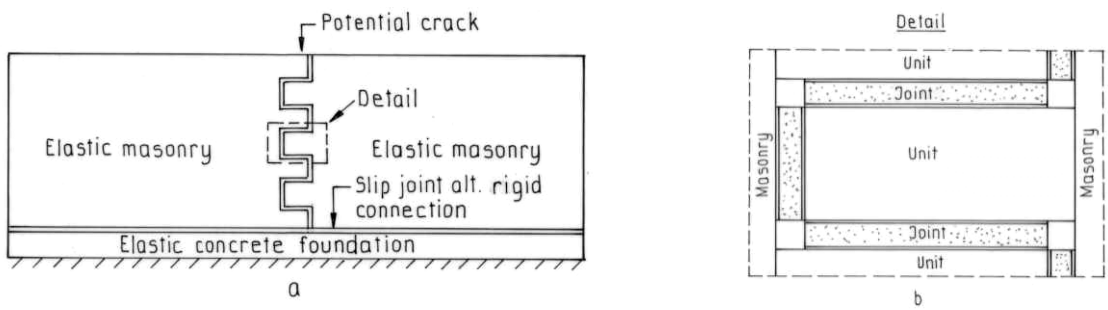

Figure 1 Massive masonry wall on concrete foundation-geometry, boundary conditions and modelling details.

Mortar joints are modelled as softening materials. Tension behaviour of the joints is simulated by spring elements, working in both compression and tension. In tension the springs exhibit softening. Crushing in compression is not included in the model. In ANSYS, combin39 elements have been employed to simulate tension behaviour of mortar joints. Shear behaviour of the joints is simulated by contact elements of type target 169 and contact 171. The contact respectively spring elements are placed in two different layers, see Fig. 2.

Bricks are modelled as linear-elastic. To circumvent computational difficulties, tensile failure of units is not included in the present model. Consequently, all cracking is concentrated to the mortar joints. In the finite element analysis, the bricks are represented by four node plane stress elements.

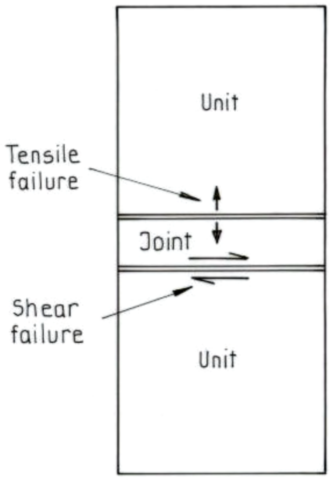

Figure 2. Detail of the joint brick boundary, with spring elements modelling tensile failure and contact elements modelling shear failure in two different layers.

Outside the zone with the pre-defined crack, the wall is modelled as a continuum with linear elastic properties. Similarly, the foundation is considered linear elastic throughout the entire simulation. Four node plane stress elements are employed to model elastic zones in both wall and foundation. For more details on the modelling see Molnár (2001) and (2004).

2.3 Material properties

In the reference case, material properties corresponding to solid clay brick masonry and a strong, cement rich mortar (described as type B) are employed. Mortar type B is composed of 100 part masonry cement (75% Portland cement and 25% filler made of grounded limestone) and 600 parts sand measured by weight. This type of masonry is the most frequently employed material combination in facades in Sweden. In the subsequent sections, this material combination is referred to as material B.

Another, often employed mortar in the Swedish masonry construction is a weaker mortar, composed of 50 parts Portland cement, 50 parts hydraulic lime and 650 parts sand measured by weight (mortar type C). The masonry resulting from combination of mortar type C with solid clay bricks is referred to as material C. The properties of materials B and C employed in the simulations are presented in Table 1.

The modulus of elasticity of the masonry composite has been determined from simulations on masonry samples with a length of 1250 mm and a height of 975 mm, with all units and joints modelled on a detailed level. As input to the modelling, experiments on bricks and mortar joints carried out in Molnár (2004) have been employed. For brick and mortar joint properties presented in Table 1, the simulations indicate that the modulus of elasticity of the masonry composite parallel to the bed joints is 5-25 % larger than the modulus of elasticity perpendicular to the bed joints. As a simplification, masonry is modelled as an isotropic material, with the modulus of elasticity parallel to the bed joints chosen as isotropic modulus of elasticity. This assumption is conservative, as, given the same imposed strain, increasing values of the modulus of elasticity result in higher stresses.

2.4 Loads

In the simulations, the self weight is applied first. Shrinkage of the wall is simulated by a negative temperature load $\alpha\Delta T$, where α is the coefficient of thermal expansion and ΔT the amount of change in temperature. The temperature of the foundation is left unchanged. The negative temperature load is increased incrementally.

Table 1. Mechanical properties of material B and C

Comp.	Property	Material B	Material C	Unit	Source
Solid bricks	Elastic modulus	13400	13400	MPa	Molnár (2004)
	Poisson's ratio	0.2	0.2	-	Estim.
	Tensile strength	2.5	2.5	MPa	Molnár (2004)
	Coeff. of thermal exp.	$6*10^{-6}$	$6*10^{-6}$	1/K	Estim.
	Density	1900	1900	kg/m^3	Molnár (2004)
Mortar joints	Secant shear stiffness	25.7	15	N/mm^3	Molnár (2004)
	Coulomb cohesion	0.40	0.25	MPa	Molnár (2004)
	Coeff. of internal friction	1.06	1.00	-	Molnár (2004)
	Coeff. of dry friction	0.98	0.90	-	Molnár (2004)
	Tensile strength	0.21	0.13	MPa	Estim.
	Mode I fracture energy	6	7.6	N/m	Estim.
Masonry	Modulus of elasticity	4000	2400	MPa	Calcul.
	Poisson's ratio	0.2	0.2	-	Estim.
	Coeff. of thermal expansion	$6*10^{-6}$	$6*10^{-6}$	1/K	Estim.
	Density	1900	1900	kg/m^3	Estim.
Concrete	Modulus of elasticity	30000	30000	MPa	Estim.
	Poisson's ratio	0.2	0.2	-	Estim.
	Coeff. of thermal exp.	$10*10^{-6}$	$10*10^{-6}$	1/K	Estim.
	Density	2400	2400	kg/m^3	Estim.

3 Simulated behaviour

In Fig. 3 the process of cracking is visualised in form of a maximum crack width versus applied shrinkage strain diagram. The value of maximum crack width is defined as the maximum horizontal deformation in the head joints situated in the mid section of the wall. In the initial phase of the loading the wall behaves linear elastically. For the chosen length to height ratio, a relatively uniform distribution of the horizontal tensile stresses in the mid section of the wall is obtained. The horizontal tensile stresses at the top of the wall correspond in this case to 75 % of the tensile stresses adjacent to the foundation. The horizontal tensile stresses in the vicinity of the restraint in the mid section of the wall can be estimated as the product between the modulus of elasticity of the masonry and the applied shrinkage strain.

Cracking starts in the head joints at the top of the wall, see point 1. Next, local cracking of the bed joints situated at the top of the wall takes place. This stage is marked by the section between point 1 and 2 in Fig. 3. Following this localization process, the crack propagates at an accelerating pace towards the foundation until all mortar joints crack. This stage is marked by point 3.

The fact that cracking of the wall starts at the top boundary, where the horizontal tensile stresses are lower than at the bottom of the wall seems unexpected. The phenomenon is explained by non-symmetrical loading of the units closest to the free edge at the top of the wall and less vertical confinement by vertical loads. Units situated at the free edge of a masonry structure are delimited by mortar joints only at one side. Horizontal loads transmitted to these units are non-symmetrical and give rise to tensile stresses in the perpendicular direction. These perpendicular stresses reduce the strength of the mortar joint and thus the strength of the masonry structure. Adjacent to a confining structure, masonry units loaded in the horizontal direction are hindered

from bending away in the vertical direction, a phenomenon which strengthens masonry. Additionally, the self weight of masonry has a strengthening effect on mortar joints.

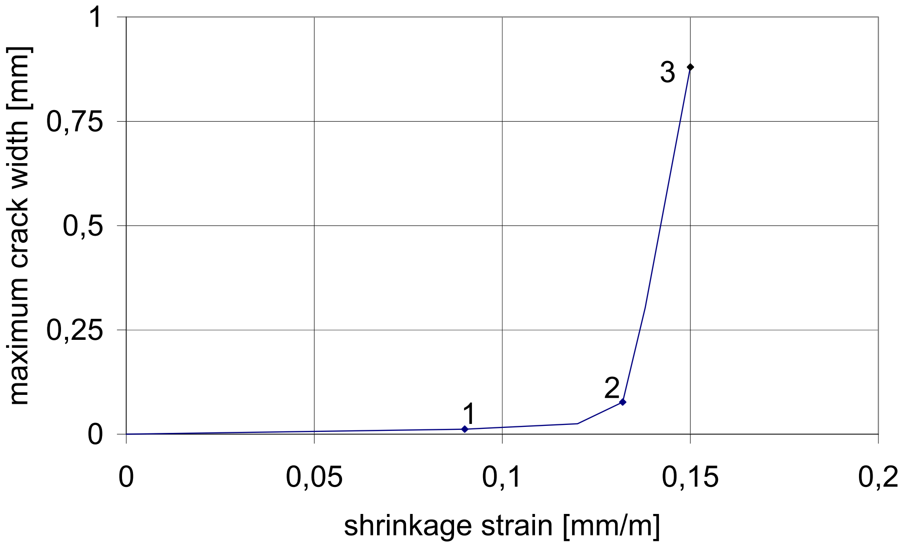

Figure 3. Cracking of masonry wall subjected to shrinkage. Wall length 12 m, height 2.4 m, material B, rigid connection between wall and foundation. 1 – onset of local cracking; 2 – onset of massive cracking; 3 – crack implying the entire height of the wall.

When failure occurs in the mortar joints, transfer of loads is still possible through dry friction in the bed joints. The magnitude of the load transmitted in this manner depends on the magnitude of the vertical load. As in the present case the vertical loads are generated by the self weight of the masonry, the horizontal loads transmitted through the crack are negligible.

After the formation of the crack the two wall parts behave in practice as free-standing walls. With increasing shrinkage the process of restrained shrinkage continues, now with conditions valid for the two wall parts. As no additional crack initiation zones are included in the model, this process is not studied in the present work.

4 Parameter study

In order to control the effects of restrained shrinkage, structural designers and material manufacturers need practical tools. In this section the influence of parameters possible to control by designers, such as the wall geometry, mechanical properties of masonry and boundary conditions is studied.

4.1 Wall geometry

Departing from the reference case in section 3, simulation of restrained shrinkage of walls with different lengths/height ratios is carried out. As the detailed modelling of the predefined crack is demanding from computational point of view, the height of the walls is kept constant, 2.4 m. Simulations are carried out with wall lengths of 7.2 and 16.8 m, corresponding to length/height ratios of 3 and 7 respectively. Material B and rigid connection between wall and foundation is considered.

The results of the simulations are presented as wall length vs. shrinkage load diagrams in Fig. 4. Critical levels of the shrinkage load are indicated as intervals delimited by the shrinkage load at crack initiation and formation of a crack comprising the entire height of the wall respectively.

Long walls crack at lower imposed shrinkage compared to short walls. The tendency in the present simulation is similar to those indicated by the analytical and numerical methods in section 1. In the wall with length of 16.8 m, the crack initiated at the top of

the wall and propagated towards the foundation, in a manner similar to that observed for the wall with length of 12 m. In the short wall (length 7.2 m) however, the crack initiated close to the base of the wall and propagated upwards.
The phenomenon is explained, by the stress distribution in the walls before crack initiation and the strengthening effect of vertical confinement on horizontal tensile strength of masonry. In short/stocky walls rigidly connected to a stiff foundation, large tensile stresses develop close to the confinement when subjected to shrinkage. Further away from the confinement, the level of tensile stresses diminishes. At length to height ratios below 2, compressive stresses exist at the top of the wall. In such walls, cracking initiates in the zone of the wall subjected to tensile stresses, i.e. close to the foundation. Crack initiation is delayed by the strengthening effect of the confinement on tensile strength of masonry. On the contrary, in long walls, an almost uniform distribution of tensile stresses is obtained. Here, cracking starts in the weakest part of the structure, which is the top of the wall.

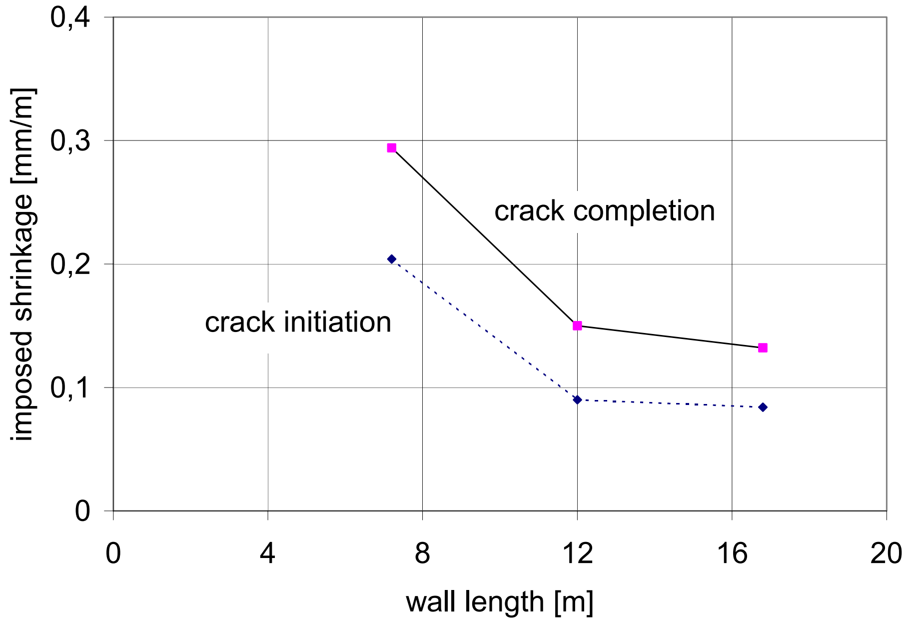

Figure 4. Influence of wall length on cracking of massive masonry walls. Shrin-kage load required to initiate resp. create a crack separating the entire wall through its vertical section. Material B, wall height 2.4 m, rigid connection between wall and foundation.

Although tensile failure of units is not included in the present model, a control shows that cracking of the 7.2 m long wall could have included tensile failure of units situated in the vicinity of the foundation. In all other cases, the walls cracked due to tensile and shear failure of the mortar joints. In the present simulations, a constant wall height was employed. By carrying out the simulations for variable heights, a more general tendency concerning crack initiation and propagation can be obtained. Considering the large computational effort required, such simulations lie beyond the scope of the present work.

4.2 Material properties

There's a large supply of different units and mortars available on the market. Is it possible for designers to influence movement joint spacing by purposive choice of masonry components? In this section, the effects of the mechanical properties of the mortar are studied.

In Section 3, the restrained shrinkage of a wall with solid clay brick units and a cement rich mortar (mortar type B) was simulated. In the present section, masonry type C built with the weaker, lime rich mortar of type C is employed. The mechanical properties of material C are shown in Tab. 1.
The simulations are carried out on walls with height 2.4 m and length 7.2 m, 12 m and 16.8 m. Rigid connection between wall and concrete foundation is assumed. From the simulations it can be seen that employing the weaker mortar C has a delaying effect on both the initiation and formation of a complete crack. Tensile stresses observed in the units are lower compared to the case with the strong mortar of type B, which indicate that tensile cracking of units is less prevalent. This is considered beneficial, as cracks involving units are considered more annoying than those arising only in the mortar-unit interface.

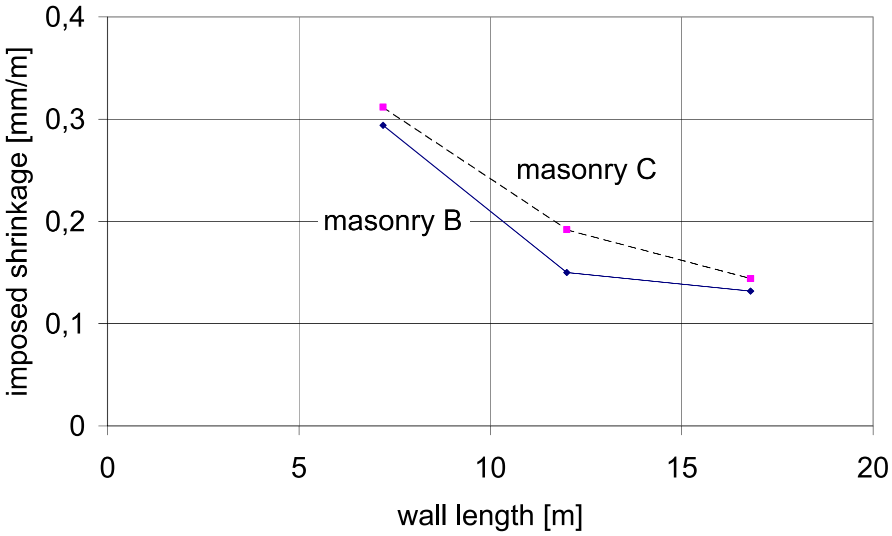

Figure 5. The effect of mortar properties and wall length on the levels of restrained shrinkage needed to create a crack through the entire vertical section in a massive masonry wall. Material B resp. C, wall height 2.4m, rigid connection between wall and concrete foundation.

The results of the simulations with walls of different lengths and material properties are presented as wall length vs. shrinkage load diagrams, together with the simulations with material B. Shrinkage loads at crack initiation and formation of a complete vertical crack are shown in Fig. 5. The simulations indicate that, by employing a weaker material, cracking of masonry walls is delayed, irrespective of wall length. Similar to the case with material B, long walls crack at lower imposed shrinkage compared to short walls. Compared to the case with the stronger material B, the increase in the capacity of the walls to resist shrinkage amounts to 10-25 %.

4.3 Slip joints

In section 3 restrained shrinkage of a masonry wall rigidly connected to the concrete foundation was simulated as a reference case. This case can be considered the most unfavourable with regard to development of tensile stresses due to a constraining foundation.
Following design recommendations, a combined slip/water-proof joint is usually placed between masonry walls resting on concrete foundations. By employing a slip joint with low cohesion, the magnitude of horizontal tensile stresses in the wall can be controlled.

As cracking of the slip joint is considered acceptable from both architectural and technical point of view, cracking of the central parts of the wall can often be avoided.
Materials employed in the slip joint are steel plates and geo-textiles prepared with water repellents. Both steel plates and geo-textiles are laid in direct contact with the concrete foundation. Next, the first layer of units are laid directly on the steel plates. In the case of geo-textiles a mortar layer is first applied, followed by a layer of units. In both cases, the boundary between the concrete foundation and the steel plate/geo-textile is considered to act as a slip joint. In spite of design recommendations, the slip joint between masonry walls and concrete foundation is some times omitted. In such cases, a cohesive boundary between walls and foundation is obtained. In this section, the effects of the mechanical properties of the boundary between the wall and concrete foundation are studied. Two cases are simulated:

- Units are laid in mortar directly on the concrete foundation.
- A proper slip joint with steel plates is employed.

In the first case the properties of the boundary are similar to those attributed to cement rich mortar joints of type B. In the case of slip joints with steel plates, the mechanical properties of the boundary are estimated from the experimental work carried out by Carlsson and Jönsson (1999). Material parameters of the boundaries employed in the simulations are presented in Tab. 2. Masonry of type B and C, with material properties shown in Tab. 1 is employed. The length to height ratio of the studied walls was set to 3, 5 and 7 respectively. The only vertical load present is the dead weight of the masonry. The imposed shrinkage load is increased until cracking of the entire slip joint occurs. Altogether 12 simulations are carried out, consisting of two material combinations, two types of slip joint and three different wall geometries.

Table 2. Mechanical properties of the boundary between wall and concrete foundation

Property	Mortar type B	Steel plate	Unit
Secant shear stiffness	25.7	25.7	N/mm^3
Coulomb cohesion	0.40	0.10	MPa
Coeff. of internal friction	1.06	1.00	-
Coeff. of dry friction	0.98	0.90	-
Tensile strength	0.21	0.05	MPa
Mode I fracture energy	6	2	N/m

In the initial phase of the loading both the slip joint and the central predefined crack behaves linear elastically. Depending on the geometry of the wall and specific material and slip joint properties, cracking initiates in the wall-foundation boundary, the central predefined crack or both. Below, the cracking process is described in a conceptual way for two typical behaviours observed in the simulations.
Behaviour type I: weak slip joint with strong masonry in the wall. The crack starts in the slip joint at the ends of the wall and propagates towards the centre of the wall. Limited cracking, principally involving the head joints, can occur in the centre of the wall. The crack develops faster in the slip joint than in the most exposed vertical section, where the development of critical tensile stresses is in this way inhibited. When the entire slip joint is cracked, transmission of constraining forces can take place only by dry friction. When dead weight of the wall is the only vertical load present, these friction forces are not sufficient to create a vertical crack in the wall. Further increase of the imposed shrinkage will not trigger the cracking in the wall.

Behaviour type II: strong, cohesive slip joint with weak masonry in the wall. Both the slip joint and the predefined crack start cracking. The crack develops faster in the pre-defined crack. A vertical crack develops through the entire wall and divides the initial wall in two halves. With increasing shrinkage load, cracking of the slip joint continues. In this stage, four active cracks exist. Two cracks propagate from the extremities towards the initial centre of the wall. Two new cracks initiate at the location of the vertical crack and propagate towards the extremities. The process is completed when the cracks meet and the two slip joints in the wall halves are entirely cracked.
In the present simulations the walls built with slip joints of steel plates exhibited only limited cracking, involving only head joints and, in a few cases, bed joints in the central pre-defined crack. By limiting the constraint between wall and foundation, a radical improvement of crack resistance of the walls could be obtained in the simulations.
To simulate a less favourable boundary condition, the steel plates in the slip joints have been replaced by mortar type B. Only walls with length to height ratio 7 built with masonry type C cracked. In the remainder of the studied cases, only limited cracking, mostly involving the head joints, was indicated by the simulations. The results of the simulations are shown in Tab. 3.

Table 3 Effect of the mechanical properties of the slip joints on cracking of walls subjected to restrained shrinkage. Simulation stopped at an imposed shrinkage ε =0.18 mm/m – slip joints cracked.

Slip joint	L/H	Masonry type B	Masonry type C
Steel plate	3	No cracks	No cracks
	5	Head joints	Head joints
	7	Head joints	Head joints
Mortar type B	3	Head joints locally	Head joints
	5	Head joints	Head joints, locally also bed
	7	Head joints, locally also bed joints	Vertical crack through the entire wall at ε =0.15 mm/m

5 Conclusions

The simulations indicate that the most effective means to delay cracking is to relieve constraints from the foundation. Constructively this is obtained by applying a slip joint consisting of e.g. steel plates between the wall and foundation. In many cases, cracking of walls can be entirely avoided in this way. The higher cohesion and friction in the slip joint, the less beneficial effect.
The simulations show that cracking of walls subjected to restrained shrinkage can in many situations be delayed by choice of weaker mortars. The effect of shifting from a strong to a weak mortar is however limited in comparison with applying a slip joint with low cohesion. Specifically for the Swedish market, a shift from the strong B mortar to the weaker C mortar, allows the spacing of the movement joints to be increased by ~10 %.
For Swedish conditions, a combined thermal and moisture movement amounting to 0.15 mm/m seems reasonable. The present work shows, that cracking in one-storey masonry facades shorter than 16.8 m provided with proper, low cohesive slip joints can be avoided. This result is in agreement with the Swedish design recommendations according to which solid clay brick facades shall be separated by movement joints every 12-16 m, see Mur90 (1990). Further work, simulating restrained shrinkage of walls longer than 16.8 m, should be carried out to assess the upper limits of movement joint spacing in masonry facades.

Acknowledgements

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