Finite Element Modelling of Unreinforced and Post-tensioned Masonry Shear Wall Assemblies

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FINITE ELEMENT MODELLING OF UNREINFORCED AND POST-TENSIONED MASONRY SHEAR WALL ASSEMBLIES

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SUMMARY

In this paper, two methods to increase the ultimate load-bearing capacity of URM shear wall assemblies are investigated. By vertical post-tensioning or by using intersecting walls as flanges for the shear walls, the stiffness and strength of the individual walls may be optimized. As a case study, a typical office building is investigated by 3D FE analysis, including nonlinear material behavior in tension (cracking/softening) and compression (hardening/softening). The influence of adding flanges and/or post-tensioning on the horizontal load-bearing capacity is evaluated.

INTRODUCTION

In the Netherlands, seismic loading is not an issue and wind loads are modest, generally less than 2 kN/m². A popular structural system to resist these lateral loads for low- and medium-rise buildings is an unreinforced masonry (URM) shear wall assembly. Such assembly consists of at least three shear walls, adequately positioned for sufficient bending and torsional stiffness of the building structure. The position of the shear walls is often dictated by the architectural floor plan of the building. The aim of the investigation described in this paper is to optimize the structural design of masonry shear wall assemblies by changing the cross-section and/or vertical post-tensioning of each individual shear wall.

The tensile strength of URM shear walls is small and unreliable, especially at the shear wall-floor interface. Therefore, URM shear walls are allowed to be cracked at ultimate limit state (ULS) and consequently the moment capacity is limited by overturning. Moreover, the horizontal load may be redistributed among the shear walls upon cracking. In this paper two methods to improve shear wall moment capacity are considered. By vertical post-tensioning or by using intersecting walls as flanges, the moment capacity of the individual walls is increased. By 3D nonlinear finite element (FE) simulations, it is investigated how this affects the shear wall assembly as a whole. These FE simulations include nonlinear behavior in tension (cracking/softening) and compression (hardening/softening).
LOAD DISTRIBUTION IN MASONRY SHEAR WALL ASSEMBLIES

The total horizontal load on a building is distributed amongst the individual shear walls. Generally, it is assumed that the floors in a multistory shear wall building act as rigid diaphragms. The shear walls are assumed to behave linear-elastically and to be coupled horizontally at floor levels. The horizontal load is distributed proportional to the stiffness of the individual shear walls. If cracking or other nonlinear behavior occurs, in any of the shear walls, the load distribution changes. This phenomenon was investigated by Sing-Sang et al. (2009) and Schermer (2005). Sing-Sang et al. (2009) investigated the load distribution in several four-story apartment buildings for wind and earthquake design loads according to the Australian Standard. They concluded that the shear walls in the investigated buildings remained linear-elastic for wind-loads prescribed in the Australian Standard. Schermer (2005) also investigated the load distribution in four-story apartment buildings, for quasi-static earthquake loading up to (numerical) failure. For increasing horizontal loads, significant redistribution of horizontal load occurred due to cracking.

BUILDING LAYOUT

The investigation described in this paper was carried out as part of a PhD project entitled *Post-tensioned shear walls of calcium silicate element masonry* by van der Meer et al. (2009). With this type of masonry, shear wall (apartment) buildings up to 12 stories can be built. In this paper, a typical 6-story office building is investigated, which has four shear walls in the z-direction. The building layout is shown in Figure 1.

![Building layout](image)

Figure 1. Building layout of typical office building (grid size = 1800 x 1800 mm²).
SHEAR WALL CONFIGURATIONS

Only two of the four shear walls in Figure 1 are considered due to symmetry, denoted Wall A and Wall B. The webs of the walls (in black) are always present, while the flanges (in gray) are optional. Self-weight of the building causes bending deformations in the same direction as wind loading from the north. Therefore, wind loading from the north is considered normative. Asymmetric wind loading is not considered. Three types of wall cross-section are investigated, namely I (rectangular), L (compressed flange) and C (two flanges). The web and flanges are assumed to be connected by overlapping of the units. The 300 mm thick shear walls are either unreinforced (U) or vertically post-tensioned (P). The prestress level of post-tensioning is equal to 0.5 MPa, which is equivalent to monostrands with a work prestress of approximately 150 kN at a spacing of 900 mm. From a total of 36 combinations, a selection of 14 was made to cover the extremes as well as the more common combinations, see Table 1. Of these, 4 are analyzed in this paper (phase 1) and 10 will be analyzed later (phase 2).

Table 1. Selected shear wall configurations for FE analysis (1 = phase 1, 2 = phase 2).

<table>
<thead>
<tr>
<th>Wall A →</th>
<th>IU</th>
<th>LU</th>
<th>CU</th>
<th>IP</th>
<th>LP</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>IU</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>LU</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>CU</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

FINITE ELEMENT ANALYSIS

A 3D finite element analysis was carried out with the software DIANA 9.4.2. Masonry is generally modeled at micro-level (separate modeling of units, joints and interfaces) or macro-level (the properties of units, joints and interfaces are smeared out over the masonry by means of homogenization techniques) (Lourenço 1996). For large models, a micro-level approach is not feasible. Therefore a macro-level approach is adopted throughout this investigation.

Due to the orientation of bed and head joints, the behavior of masonry is generally orthotropic, especially regarding tensile strength parallel and perpendicular to the bed joints. In this investigation, a tensile strength perpendicular to the bed joints of 0.2 MPa was adopted for the shear walls as well as the wall-floor interface. To take orthotropy into account, a multiface plasticity model developed by Lourenço (1995) was used, which consists of a Rankine-type yield surface for tension and a Hill-type yield surface for compression. The model allows for different tensile and compressive strengths in x- and y-directions. The model of Lourenço (1995) is available as FORTRAN code, which can be used as a User-Supplied Subroutine in DIANA. For this investigation, a version of the code from the report of Sing-Sang et al. (2005) was used. This nonlinear material model requires the input of 16 parameters, which are given in Table 2. To take material strength variation into account, a random strength variation of ± 5% was assigned to the elements. The local x-axis is perpendicular to the bed joints (coincides with the global y-axis, see Figure 1). The local y-axis is parallel to the bed joints and coincides with the global x- or z-axis depending on the wall orientation.
Table 2. Material properties for Rankine-Hill plasticity model.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Meaning</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{tx}$</td>
<td>Tensile strength in local x-direction (global y-direction)</td>
<td>0.2</td>
<td>MPa</td>
</tr>
<tr>
<td>$G_{tx}$</td>
<td>Tensile fracture energy in local x-direction</td>
<td>1.696 $\times$ 10^{-3}</td>
<td>Nmm/mm^2</td>
</tr>
<tr>
<td>$f_{ty}$</td>
<td>Tensile strength in local y-direction</td>
<td>1.0</td>
<td>MPa</td>
</tr>
<tr>
<td>$G_{ty}$</td>
<td>Tensile fracture energy in local y-direction</td>
<td>8.48 $\times$ 10^{-2}</td>
<td>Nmm/mm^2</td>
</tr>
<tr>
<td>$f_{mx}$</td>
<td>Compressive strength in local x-direction</td>
<td>10.0</td>
<td>MPa</td>
</tr>
<tr>
<td>$G_{mx}$</td>
<td>Compressive fracture energy in local x-direction</td>
<td>3.82</td>
<td>Nmm/mm^2</td>
</tr>
<tr>
<td>$f_{my}$</td>
<td>Compressive strength in local y-direction</td>
<td>10.0</td>
<td>MPa</td>
</tr>
<tr>
<td>$G_{my}$</td>
<td>Compressive fracture energy in local y-direction</td>
<td>3.82</td>
<td>Nmm/mm^2</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Parameter to relate tensile and shear strength</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>$\alpha_g$</td>
<td>Parameter represents Rankine plastic flow ($\alpha_g = 1.0$)</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Parameter for biaxial compressive strength</td>
<td>-1.0</td>
<td>-</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Parameter to relate compressive and shear strength</td>
<td>3.0</td>
<td>-</td>
</tr>
<tr>
<td>$\kappa_p$</td>
<td>Equivalent plastic strain at compression peak stress</td>
<td>2.83 $\times$ 10^{-3}</td>
<td>-</td>
</tr>
<tr>
<td>$h$</td>
<td>Crack bandwidth ($h=\sqrt{2A_{element}}$)</td>
<td>424.3</td>
<td>mm</td>
</tr>
<tr>
<td>$E_x$</td>
<td>Young’s modulus in local x-direction</td>
<td>15.0</td>
<td>GPa</td>
</tr>
<tr>
<td>$E_y$</td>
<td>Young’s modulus in local y-direction</td>
<td>15.0</td>
<td>GPa</td>
</tr>
</tbody>
</table>

A graphical representation of the Rankine-Hill model, using the material parameters from Table 2, is given in Figure 2. The contour-plot shows the Rankine-Hill yield criterion for various values of the shear stress $\tau_{xy}$. The stress-strain diagram, also based on the parameters from Table 2, includes tension softening, compression hardening and compression softening, and is shown in Figure 3. Plastic strain develops when the tensile strength is exceeded or when the compressive stress exceeds one-third of the compressive strength. During a nonlinear analysis, DIANA evaluates for every integration point whether the yield criterion is exceeded. If so, an Euler implicit backward return-mapping scheme is used to update the stress vector. Also, the tangent stiffness matrix of the integration point is updated. The updated properties of the integration point are returned to the global FE analysis for each iteration within every load step. More detailed information may be found in Lourenço (1995).

Figure 2. Graphical representation of Rankine-Hill model (for parameters of Table 2).
In addition to the material properties in Table 2, linear-elastic properties were given to both concrete and calcium silicate element (CASIEL) masonry. Concrete was given a Young’s modulus of 30.0 GPa and a Poisson’s ratio of 0.2. CASIEL masonry was given a Young’s modulus of 15.0 GPa and a Poisson’s ratio of 0.2. As a simplification, the unbonded post-tensioning was applied as an external load. Therefore, no material and physical properties of the post-tensioning steel were required.

The shear walls and floors were modeled using four-node curved shell elements (Q20SH) with a mesh size of 300 x 300 mm². The local x- and y-axes of the shear wall elements were oriented according to their orthotropic behavior. The load-bearing walls (white walls in Figure 1) were modeled as truss elements (L2TRU) without any contribution to the lateral stiffness. The thickness of the shell elements is equal to the shear wall thickness. The curved shell elements have 2x2 integration points in their area and 3 integration points across their thickness (by default). The integration schemes are Gauss for integration over the area and Simpson for integration over the thickness.

Different boundary conditions were applied to the FE model. The floor edges in the plane of symmetry were fixed in x-direction and prevented to rotate about y- and z-axes. The concrete foundations below the shear walls were fixed in x-, y- and z-directions. The load-bearing walls were supported in y-direction below the ground floor. The floors, shear walls and load-bearing walls were given a distributed mass. The floor density was increased to incorporate the weight of its non-structural part. The wind load and self-weight of the facade were introduced as line loads via the floors into the structure. To achieve this, the floors were extended 300 mm over the walls, because introduction of these loads directly into the wall-floor interface led to peak stresses, which gave problems in the nonlinear analysis. The facade load was divided into two parts, one with and one without window openings. When flanges were not modeled, extra truss elements (load-bearing walls) were inserted at these locations. The physical properties and loads for the FE model are shown in Table 3.

For each of the FE models, a linear-elastic analysis was performed prior to a nonlinear analysis. In the nonlinear analysis, the vertical load was applied first, subsequently the post-tensioning (if applicable) and finally the horizontal load was increased. The full Newton-Raphson method was used to obtain a converged solution for each load step. The energy norm was used as a convergence criterion and a line search technique was applied to increase convergence speed. The energy norm is the variation of the internal strain energy in the current iteration divided by the variation of internal strain energy at the beginning of the load increment. Convergence was accepted when the energy norm was less than $10^{-4}$. When
convergence could not be achieved within 30 iterations, the analysis was still continued,
because in some cases the next load step was able to achieve convergence. Non-converged 
load steps were evaluated carefully afterwards. Since the wind load was prescribed, an arc-
length technique called indirect displacement method was used. This method is available in 
DIANA for cases where it is convenient to prescribe a load while displacement-control would 
be preferable. Second-order effects were neglected.

Table 3. Physical properties and loads for FE model.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Story height</td>
<td>3600</td>
<td>mm</td>
</tr>
<tr>
<td>Number of stories</td>
<td>6</td>
<td>-</td>
</tr>
<tr>
<td>Floor thickness (structural)</td>
<td>260</td>
<td>mm</td>
</tr>
<tr>
<td>Floor thickness (non-structural)</td>
<td>50</td>
<td>mm</td>
</tr>
<tr>
<td>Foundation beam / roof beam thickness</td>
<td>300</td>
<td>mm</td>
</tr>
<tr>
<td>Foundation beam height</td>
<td>900</td>
<td>mm</td>
</tr>
<tr>
<td>Roof beam height (for introduction of post-tensioning)</td>
<td>300</td>
<td>mm</td>
</tr>
<tr>
<td>CASIEL wall thickness</td>
<td>300</td>
<td>mm</td>
</tr>
<tr>
<td>Area of load-bearing walls</td>
<td>214 x 600</td>
<td>mm²</td>
</tr>
<tr>
<td>Concrete density (corrected to include non-structural part)</td>
<td>2860</td>
<td>kg/m³</td>
</tr>
<tr>
<td>CASIEL density</td>
<td>2200</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Gravity acceleration</td>
<td>9.81</td>
<td>m/s²</td>
</tr>
<tr>
<td>Facade load (facade with window openings)</td>
<td>4.9</td>
<td>N/mm</td>
</tr>
<tr>
<td>Facade load (facade without window openings)</td>
<td>7.2</td>
<td>N/mm</td>
</tr>
<tr>
<td>Wind load (1 kN/m²)</td>
<td>3.6</td>
<td>N/mm</td>
</tr>
</tbody>
</table>

RESULTS AND DISCUSSION

All models were evaluated in the ULS. The material properties from Table 2 are design values 
for the material CS44 (calcium silicate element masonry with a characteristic unit 
compressive strength of 44 MPa). The idea is that the ULS calculations give an indication of 
the maximum capacity and the horizontal load redistribution with increasing horizontal load. 
At a later stage the maximum horizontal deflection and the occurrence of cracking could be 
checked in the serviceability limit state (SLS), using representative material properties. This 
paper focuses solely on the ULS calculations. Prior to nonlinear analyses, linear-elastic 
analyses were carried out, which will be discussed first.

LINEAR-ELASTIC ANALYSIS

The linear-elastic analysis was carried out to derive the following results:

- Average vertical stress in the shear walls due to dead load;
- Horizontal load distribution between wall A and wall B.

These results were derived by evaluation of the horizontal and vertical reaction forces. All 
results are given for the FE model (half of the building). The total horizontal (wind) load 
equaled 255.4 kN (1 kN/m²). The total vertical load was approximately 14400 kN for 
combination C-C and 13800 kN for combination C-I. The linear-elastic results are given in 
Table 4. If wall A and B have the same cross-section, the wind-load is distributed 
approximately fifty-fifty. A part of the vertical load is carried by columns (load-bearing 
walls). In the FE model, extra columns were inserted when flanges were absent, to simulate a 
realistic distribution of the vertical load. Comparison of the average vertical stress in wall A
and B, shown in Table 4, indicates that the average vertical stress in wall A is lower than in wall B. This is due to the position of wall A near the facade. Furthermore, the vertical stress is lower when the cross-sectional area increases. The eccentricity of the floors with respect to the shear walls induces flexure in the shear walls due to self-weight of the building. Therefore, the vertical stresses given are averaged values. The axial stress-strength ratio between 13 and 17 % (the vertical stress from Table 4 divided by the compressive strength from Table 2) indicates that post-tensioning is advantageous for these shear walls.

### Table 4. Horizontal and vertical load distribution and vertical stress (linear-elastic results).

<table>
<thead>
<tr>
<th>Wind load (%)</th>
<th>Vertical load (%)</th>
<th>Vertical stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>C-C</td>
<td>49.2</td>
<td>50.8</td>
</tr>
<tr>
<td>C-I</td>
<td>60.7</td>
<td>39.3</td>
</tr>
</tbody>
</table>

### NONLINEAR ANALYSIS

In the nonlinear analysis, the following aspects were analyzed:

- The horizontal load-displacement diagram;
- The development of cracks and plasticity (nonlinear compression);
- The horizontal load redistribution;
- The axial stress at the bottom of the wall for various load levels;
- The shear stress at the bottom of the wall for various load levels.

### LOAD-DISPLACEMENT DIAGRAM

The north edge of the plane of symmetry at roof level was taken as a reference point for horizontal displacement. The horizontal load was equal to the load factor in the nonlinear analysis, since the prescribed load equaled a wind load of 1 kN/m². The load-displacement diagram for combinations C-C and C-I are shown in Figure 4. The initial displacement is due to eccentricity of the weight of the floors with respect to the shear walls. The ultimate loads reached in the analyses were higher than shown in Figure 4. In fact, they were even higher than the theoretical upper limit based on equilibrium of the bottom section of the shear wall. This was due to development of tensile stresses in the URM columns (load-bearing walls) in the north facade, which were modeled linear-elasticantly. These tensile stresses would cause cracking, which should not be allowed, especially when the wind-load on the facade is introduced into the floors via these columns. Therefore, in this paper, the ULS is determined by the occurrence of tensile stresses in the columns instead of complete failure by overturning of the shear walls.

![Figure 4. Load-displacement for unreinforced and post-tensioned shear wall combinations.](image-url)
CRACKING, PLASTICITY AND DEFORMED SHAPE

The user-supplied subroutine has four yield combinations for the material x- and y-directions, namely tension (-), compression ( | ), tension-compression and tension-tension, see Figure 2. The symbols in parentheses can be plotted on the deformed mesh. A ‘-’ indicates that an integration point is cracked and softening in tension and a ‘|’ indicates that the compressive stress exceeds one-third of the compressive strength (onset of nonlinear behavior in compression). The tension-compression and tension-tension regimes are not relevant for the analyses discussed in this paper. The occurrence of cracking and plasticity is plotted on the deformed shape of the bottom story of wall B for combination CP-CP, see Figure 5.

![Figure 5](image)

Figure 5. Cracking (-), plasticity ( | ) and deformed shape (scale factor = 200) of wall B in CP-CP (bottom story) for increasing load factor ( x 1.0 kN/m²).

HORIZONTAL LOAD REDISTRIBUTION

The horizontal load is distributed amongst walls A and B. The linear-elastic load distribution was evaluated previously, see Table 4. Upon nonlinear behavior, redistribution of the horizontal load can occur. This was investigated by comparing the sum of horizontal reactions of the individual shear walls. Horizontal reactions occurring due to self-weight (some torsion occurred) were subtracted from those due to the horizontal wind load. Results are shown in Figure 6 for shear wall combinations C-C and C-I. Up to the point of cracking, the linear-elastic load-distribution is maintained. After cracking, the load is redistributed from wall A to wall B, for both combinations. For combinations C-C the redistribution is minor, while for combination C-I, more significant redistribution occurs.

![Figure 6](image)

Figure 6. Horizontal load redistribution for shear wall combinations C-C and C-I.
AXIAL AND SHEAR STRESSES

For various load levels, axial and shear stresses were evaluated at the bottom layer of shear wall elements. For each element, the Gaussian results were averaged over a top, middle and bottom plane. The stresses in the web were evaluated for the middle plane, see Figure 7, while the stresses in the flanges were evaluated for the planes corresponding to the outer fiber of the cross-section, see Figure 8. The axial stresses are as expected, clearly showing the capability of the FE model to simulate flexural cracking. The shear stresses in Figure 7b are not as expected, but they can be explained by the difference in lateral expansion between the masonry wall and concrete floor. At half of the first story height, shear stresses followed the expected parabolic course along the web.

CONCLUSIONS AND RECOMMENDATIONS

In this paper, results of 3D nonlinear FE analysis of a typical six-story office building were presented as a case study. Conclusions cannot be generalized to all shear wall buildings. Nevertheless, interesting observations were made:

- Post-tensioning increases the horizontal load capacity considerably, to approximately 146% for this case study;
• Comparison of shear wall combinations C-C and C-I shows that adding flanges to wall B increases the horizontal load capacity to approximately 134 %;
• The results shown in this paper are up to the load level at which the north-facade columns exhibit tensile stresses. The shear walls have not reached their theoretical ultimate capacity at this stage. It is interesting to see that the columns influence the ULS of the building, even though they do not contribute to the lateral stiffness. If the shear walls were analyzed as isolated structural elements, this result would not have been found;
• Cracking occurred at the wall-floor interfaces, especially at the ground floor level. Plasticity concentrated in the compressed zone at the base of the shear wall. This agrees with flexural behavior of the shear wall;
• The deformed shape of the shear walls in this case study can be described as cantilevered. Clearly, a shear wall test with fixed-fixed boundary conditions is not adequate to validate the flexural behavior observed in these FE analyses;
• In combinations with shear walls of the same cross-section (C-C), the load redistribution was minor, ± 2.2 - 3.6 %. For combination C-I, the load redistribution was more significant, ± 7.6 – 19.3 %. More redistribution occurred for unreinforced combinations compared to post-tensioned combinations. Only for combination CU-IU, the redistribution exceeded the 15 % stated in Eurocode 6 (§ 5.5.3).

Further research will include the shear wall combinations of phase 2 as indicated in Table 1.

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REFERENCES


