Numerical Study Of Pier-Wall Connections In Typical Dutch Urm Buildings

Fusco, Daniela; Messali, F.; Rots, J.G.; Addessi, Daniela; Pampanin, Stefano

Publication date
2021

Document Version
Final published version

Published in
12th International Conference on Structural Analysis of Historical Constructions

Citation (APA)
Fusco, D., Messali, F., Rots, J. G., Addessi, D., & Pampanin, S. (2021). Numerical Study Of Pier-Wall Connections In Typical Dutch Urm Buildings. In P. Roca, L. Pelà , & C. Molins (Eds.), *12th International Conference on Structural Analysis of Historical Constructions : SAHC 2021, Online event, 29 Sep - 1 Oct, 2021* (pp. 2217-2228). International Centre for Numerical Methods in Engineering, CIMNE. http://congress.cimne.com/SAHC2020/frontal/doc/Ebook_SAHC2020.pdf

NUMERICAL STUDY OF PIER-WALL CONNECTIONS IN TYPICAL DUTCH URM BUILDINGS

DANIELA FUSCO[1*], FRANCESCO MESSALI[2], JAN G. ROTS[2], DANIELA ADDESSI[1] AND STEFANO PAMPANIN[1]

[1]Sapienza Università di Roma
Dipartimento di Ingegneria Strutturale e Geotecnica
Via Eudossiana 18, 00184 Rome, Italy
e-mail: d.fusco1994@gmail.com, {daniela.addessi, stefano.pampanin}@uniroma1.it
(*corresponding author)

[2] Delft University of Technology
Department 3MD
Building 23, Stevinweg 1, 2628 CN Delft, The Netherlands
e-mail: {F.Messali, J.G.Rots}@tudelft.nl

Keywords: Glued Connection, Masonry, Seismic, Modelling

Abstract. *In recent years, the seismic risk in the north of the Netherlands has increased due to gas extraction. Since 2014, the Delft University of Technology started a research program to assess the seismic response of unreinforced masonry (URM) buildings. The Dutch URM buildings are characterized by slender piers and transverse walls. In common practice, the connections between piers and transverse walls are often modelled as rigid, but in real structures these connections may exhibit different behaviour. Especially, since the 1980s, calcium silicate element masonry has been commonly used in Dutch buildings, and vertical continuous joints are present between transverse walls. For this reason, it appears essential to assess the connection strength properties, since its failure can significantly reduce the seismic performance of the entire structure. The first part of this work investigates and compares different numerical approaches to describe the nonlinear behaviour of masonry under lateral loads, simulating seismic action. The second part specifically focuses on the critical issues related to the modelling of vertical connections of Dutch URM buildings. A sensitivity study of the frictional parameters is performed to analyze the influence of the strength of the glued connection on the global response of the URM structure.*

1 INTRODUCTION

Typical Dutch Unreinforced Masonry (URM) terraced houses are composed of façades with large openings and slender piers, connected at corners with long transversal walls. The seismic behaviour of the entire structure is determined by the quality of the connections between the transverse wall and piers. In buildings constructed before 1980, which make use of calcium silicate (CS) bricks, the connection is guaranteed by the interlocking of the units. In the 1980s, large CS elements started being used in order to accelerate the construction process, and the connections between the walls and the piers were provided by means of vertical glued joints with steel ties at the bed-joint level. If this connection fails, the vertical glued joint may slide and open, and the capacity of the structure suddenly decreases [1].

In the past, Raijmakers and Van der Pluijm [2] conducted an experimental research to analyse the failure of the vertical glued connection at the structural element level, i.e. considering only a wall-pier system. Later on, the increasing induced seismicity in the Groningen area has led to a strong interest of Dutch scientific community to the assessment of the seismic vulnerability of Dutch URM structures. Delft University of Technology tested two full-scale two-storey buildings to evaluate the seismic capacity of these structures [3,4,5,6,7].

In recent years, several methods have been developed to analyse and predict the seismic performance of URM buildings [8,9,10]. The nonlinear analysis through the finite element method (FEM) is one of the most common approaches to calculate the seismic capacity of the structure in terms of ultimate displacement and maximum strength. In FEM models, the nonlinear behaviour of the masonry is assigned to the constitutive law of the finite elements. Different approaches can be distinguished and categorised according to the scale of the analysis [11,12,13]. One of the most adopted criteria classifies these as micromechanical, macromechanical, macro-element and multiscale models [14].

This work investigates the appropriate numerical modelling of the vertical glued connections response, in order to study the influence of these connections on the global seismic capacity of the structure. To reach this goal, the first part of the paper focuses on the FE macromechanical modelling of the nonlinear behaviour of masonry structures under lateral loads, simulating seismic actions. In particular, the use of anisotropic or isotropic constitutive laws for masonry is discussed in Section 2. Subsequently, Section 3 compares two different constitutive laws applied in the model to reproduce the possible failure of the vertical glued connection.

2 FINITE ELEMENT MODELS FOR MASONRY

This section focuses on two finite element (FE) models used to assess the seismic capacity of URM structures. In common practice, two different strategies are used to model the nonlinear behaviour of the material: discrete and smeared cracked models [1,12,15]. In discrete crack models, the nonlinear behaviour is lumped in the interface elements located where the crack or the sliding may occur. Instead, in smeared crack models, the crack is smeared out over the finite element and it may occur in any direction. In this work, two different material constitutive laws are compared: the Total Strain Crack Model and the Engineering Masonry Model, both smeared crack model implemented in DIANA FEA [15].

2.1 Constitutive laws

The Total Strain Crack Model (TSCM) assumes a tensile softening behaviour governed by the Mode I fracture energy [1,15]. The compressive behaviour can be defined on the basis of several constitutive functions that describe the hardening-softening compression curves. Regarding the shear behaviour, DIANA permits to fix a reduced shear stiffness after cracking. Some limits of the Total Strain Crack Model are worth to mention, i.e.: the anisotropy of the material is not considered, the shear failure is not distinguished from the tensile failure and the energy dissipation under cyclic loading is underestimated, especially in case of shear failure.

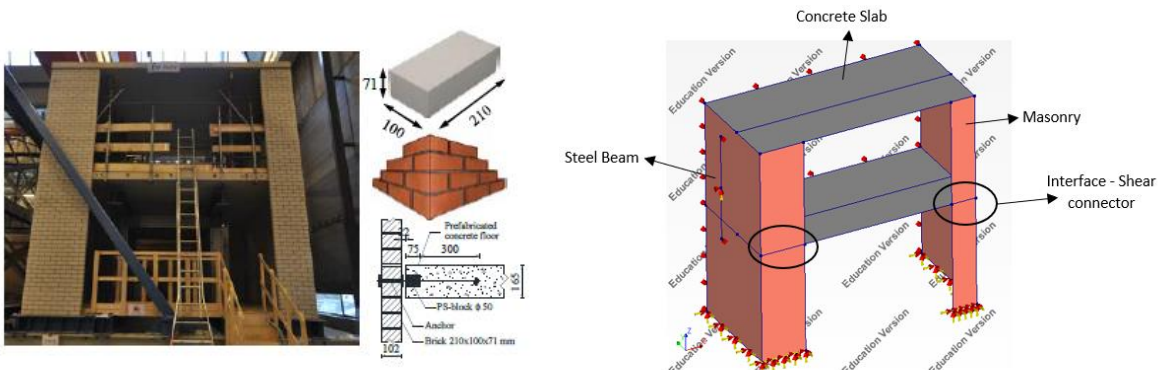
The Engineering Masonry Model (EMM) permits to overcome the mentioned shortcomings of the Total Strain Crack Model. In particular, on the one hand, the Engineering Masonry Model considers the anisotropic property of the masonry and differentiates between tensile and shear failure (implementing the shear failure Coulomb's criterion), and on the other hand it provides a more realistic estimate of the dissipated energy for cyclic analyses in case of shear failure of a pier [15,16].

2.2 Numerical Application: Modelling of a tested two-storey masonry structure

The two constitutive laws described in the previous section are adopted to perform the numerical analysis of the nonlinear structural response of an experimental masonry structure. The aim is to validate the capability of the employed FE procedures to accurately describe the activation and evolution of the nonlinear mechanisms in masonry structures, possibly leading to their collapse.

One of the two two-storey masonry structures tested in 2015 at Delft University of Technology (TU Delft) is considered [3,4]. The specimen, illustrated in Figure 1 (a), is a full-scale masonry assemblage representative of the load-bearing structure of a typical two-storey terraced house built in the Groningen province in the period 1960-1980. These buildings are characterized by small CS bricks and running bond pattern.

The masonry house is modelled with either shell elements or solid elements. Since the structure is symmetric, half of the structure is considered to reduce the computation effort. The slab of the second floor lies up on the walls and the piers via mortar joint, thus this connection can be assumed strong enough to consider shared nodes. The anchors between the first floor and the piers are used to retain any out-of-plane movement of the piers and they are not able to transfer any significant shear load. For this reason, they are modelled with interface elements having non-zero stiffness only in the direction orthogonal to the piers. During the experimental test, a cyclic loading was applied through four actuators coupled in order to maintain the forces equal at the two floors. This loading condition was simulated with a displacement controlled analysis, by adding an auxiliary rigid steel beam with appropriate restrains.



(a) (b)

Figure 1: (a) Specimen and construction detail of a typical two-storey terraced house [4] and (b) Simplified modelling schematic.

Three different FE models of the building are implemented and analyzed via the code

DIANA FEA, namely:

- TSCM-Shell: model with shell FEs adopting the Total Strain Crack Model;
- TSCM- Solid: model with solid FEs adopting the Total Strain Crack Model;
- EMM-Shell: model with shell FEs adopting the Engineering Masonry Model;

The material parameters used for the three models, calibrated and adapted on the basis of the material tests performed and reported in [3] and [7], are summarized in Tables 1 and 2.

Table 1: Masonry mechanical parameters: Engineering Masonry Model

Elastic Modulus perpendicular to head joint	E_x	2212	MPa
Elastic Modulus perpendicular to bed joint	E_y	3264	MPa
Shear Modulus	G	1306	MPa
Mass Density	ρ	1805	kg/m^3
Tensile strength normal to bed joint	f_{ty}	0.19	MPa
Minimum strength head-joint	f_{tx}	0.38	MPa
Tensile fracture Energy	G_{ft}	0.0127	N/mm
Angle between stepped diagonal crack and bed joint	θ	0.792	rad
Compression strength	f_c	5.8	MPa
Fracture Energy in compression	G_{fc}	17.4	N/mm
Factor to strain at compressive strength	n	5	
Unloading Factor	λ	0	
Friction angle	γ	0.406	rad
Cohesion	f_{vo}	0.14	MPa

Table 2: Masonry mechanical parameters: Total Strain Crack Model

Elastic Modulus	E	3264	MPa
Poisson's ratio	ν	0.16	
Shear Modulus	G	1306	MPa
Mass Density	ρ	1805	kg/m^3
Crack Orientation		Rotating	
Tensile Curve		Linear- crack energy	
Tensile strength	f_t	0.19	MPa
Tensile fracture Energy	G_{ft}	0.0127	N/mm
Crack bandwidth specification		Rots	
Compression Curve		Parabolic	
Compression strength	f_c	5.8	MPa
Fracture Energy in compression	G_{fc}	17.4	N/mm

A pushover analysis is performed and the capacity curves obtained with the different models are compared with the backbone curve derived from the experimental results [4,5] in Figure 2 (a). The elastic stiffness of the models coincides with that of the experimental curve. After the peak, the stiffness of the structure is significantly reduced, due to the development of the rocking mechanism. During this phase, the cracks are located at the bottom and top of the pier sections, where tensile strains occur. The rocking mechanism occurs in every model, but the maximum base shear is higher in the EMM-Shell model than in the others. The main differences in the results obtained with the different constitutive models emerge in the post-peak phase. Indeed, the curves evaluated with the TSCM models suddenly decrease resulting in severe softening branches, whereas the EMM model gives a more gradual stiffness and strength degradation. This depends on the observed prevailing failure mechanism. In particular, the loss of capacity in the experimental test was determined by the diagonal cracking of the wide pier (Figure 2 (b)). The same diagonal crack was observed for both the TSCM models (Figure 2 (d) and Figure 3(e)) although this leads to a more brittle failure than that observed in the experiment. On the other hand, the use of EMM constitutive law switches the failure to the toe-crushing of the wide pier, as shown in Figure 2 (c). The use of solid or shell elements affects only the peak strength of the structure, but not the type of failure. The difference also in terms of strength is rather limited for positive displacements, and more significant (yet not very large) for negative displacements.

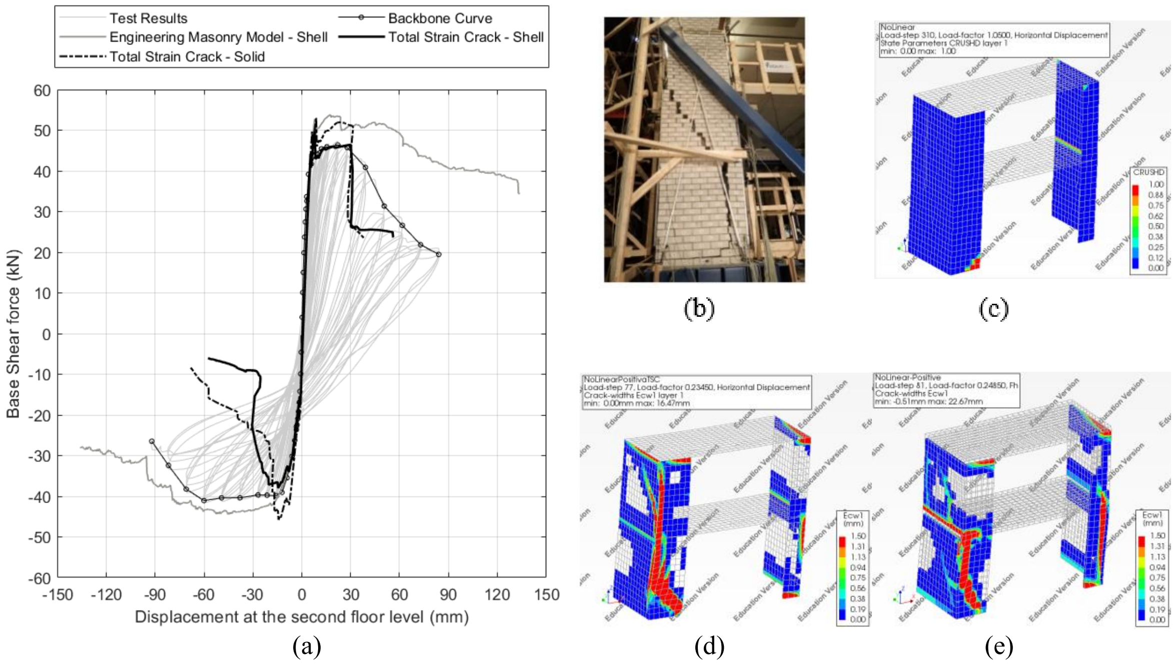


Figure 2: (a) Comparison of the capacity curves: Experimental results, EMM Shell model, TSCM Solid and TSCM Shell. Failure mechanism for each model: (b) Experimental test [Diagonal cracking] [4], (c) EMM Shell model [Toe-crushing], (d) TSCM Shell model [Diagonal cracking] and (e) TSCM Solid model [Diagonal cracking]

3 FINITE ELEMENT MODELLING FOR GLUED VERTICAL CONNECTIONS

As previously mentioned, the seismic capacity of a typical Dutch URM structure is affected by the strength of the vertical connection between the main wall and piers. This was clearly shown by the experimental research performed by Raijmakers and Van der Pluijm [2], which consisted in applying a horizontal load to a U-shaped construction, composed of the main wall and two perpendicular piers. The wall and the pier are composed of calcium silicate element masonry. The results of the test show three possible types of failure mechanisms (Figure 3): the rocking mechanism of the whole structure (a), the diagonal cracking/compression failure of the pier (b), and the shear failure of the wall-pier connection (c). The mechanism (a) can be preliminary to the other two. The mechanism (b) is frequent in the toothed connection and the mechanism (c) occurs for weaker connection type, as in the case of vertical glued connection. The shear failure of the vertical connection leads to a sudden strength reduction, which affects the seismic capacity of the entire structure [1]. In order to consider this mechanism in the numerical modelling, the vertical glued connection is modelled through interface elements with nonlinear behaviour. Two different constitutive laws, both implemented in DIANA FEA, are considered for the interface: Nonlinear Elastic Model and Coulomb Friction Model.

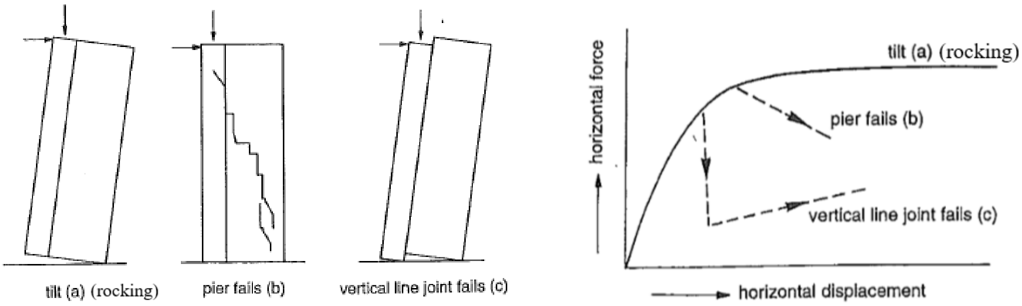


Figure 3: Possible failure mechanisms of a U-Shaped wall [1]

3.1 Constitutive laws for interface elements

The Coulomb friction model for interface elements is based on the Mohr-Coulomb plasticity model, defined by cohesion, friction, dilatancy angle and Mode II energy fracture. In this model, the shear failure depends on the normal stress acting on the interface. The coupled behaviour increases the complexity of the model making more unstable the numerical solution. More details on this type of modelling approach are available in literature [1,12,15].

Alternatively, the nonlinear elastic constitutive law is defined by the relative displacement-traction diagrams both in the normal direction and in the shear direction [15]. In contrast to the Coulomb friction model, the axial (orthogonal to the interface) and shear behaviour are decoupled. The diagonal tangent stiffness matrix with decoupled terms improves the robustness of the model, facilitating the convergence of the analysis.

3.2 Numerical Applications: Modelling at the element and structural level

The modelling in DIANA FEA of the vertical glued connection is analyzed first at the structural element level, i.e. considering only a wall-pier system tested by Raijmakers and Van der Pluijm [2], and then at the structural level on the full-scale two-storey building described in the previous paragraphs.

At the structural element level, two different models, adopting plane stress (2D-Model) and solid elements (3D-Model), were used. The Calcium Silicate masonry is modelled with linear elastic elements. No-tension behaviour was assigned at the base joint, and boundary interface elements were applied between the masonry elements and the fixed supports. An equivalent vertical load, which represents the stabilizing moment given by the weight of the floor, was applied at the top of the wall. The prescribed displacement is applied on the top of the wall, thus performing a displacement-control analysis.

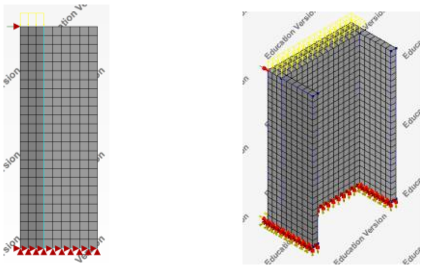
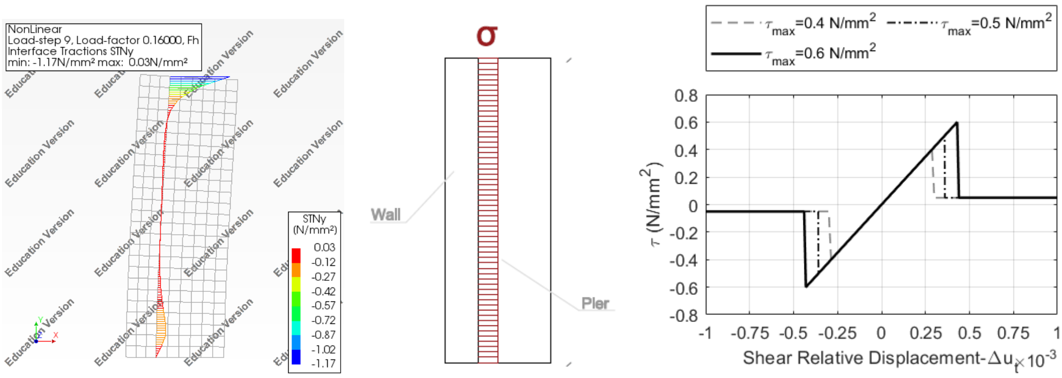


Figure 4: Model of the pier-main wall in DIANA FEA: 2D-Model [Left] and 3D-Model [Right]

The vertical glued connection is modelled through interface elements between the wall and the pier. As previously mentioned, the real behaviour of the interface is governed by the Coulomb friction failure criterion, where the shear strength depends on the normal stress, which varies along the connection, as illustrated in Figure 5 (a). Instead, the nonlinear constitutive law assumes that the shear capacity is equal for each point of the interface. This assumption corresponds to assume a constant normal stress along the interface, as illustrated in Figure 5 (b).

The material parameters for masonry and interface elements are obtained from the literature [1] and summarized in Table 3.

Alternatively, a nonlinear elastic constitutive law is assigned to the structural interface elements, and the relative displacement-shear stress diagrams is defined as illustrated in Figure 5 (c). Three different maximum shear stresses were considered, τ_{max} = 0.4 N/mm^2, 0.5 N/mm^2, 0.6 N/mm^2 (which corresponds to assume a constant distribution of the normal stresses equal to σ=0 N/mm^2, σ=0.13 N/mm^2 and σ=0.27 N/mm^2, respectively).



(a) (b) (c)

Figure 5: Normal stress along the connection: Coulomb Friction Model (a) vs with Nonlinear Elastic Model (b). Relative Displacement - Shear Stress Diagram (c)

Table 3: Vertical interface-Coulomb Friction Model properties [1]

Vertical Joint	Normal Stiffness	k_n	3125	MPa
(Coulomb Friction)	Shear Stiffness	k_t	1395	MPa
	Mode II fracture energy	G_f^{II}	0.05	J/m^2
	Cohesion	c_u	0.4	N/mm^2
	Angle of friction	tanφ	0.75	-

The Coulomb model for the interface generally requires a high computational effort, and it is excessively time-consuming. The analysis converges until the occurrence of the shear failure along the vertical interface, when the shear stress of several nodes reaches the Coulomb strength domain boundary. At that point, the sudden propagation of the crack along the interface leads to instability of the numerical solution and, for the Newton-Raphson iterative method, to lack of convergence of the analysis. Eventually, the Secant (Quasi-Newton) method with a high number of steps was used to follow the post-peak behaviour. The disadvantage is that the initial stiffness reduction, caused by the rocking, is not captured and the curve maintains an elastic behaviour up to the failure of the interface. The analysis performed by Rots [1] using the arc-length method provides a capacity curve similar to that obtained by the experiment. With this method, it is possible to maintain a stable solution during the snap-back, which represents the propagation of the crack along the vertical interface. Figure 6 illustrates the comparison between the three iterative methods. No difference between the modelling with solid elements and plane stress elements is found, except in terms of the computational effort. Besides, it was more difficult to reach convergence in the model with solid elements than in that with plane stress elements.

In case of nonlinear elastic constitutive law, the analysis with Newton-Raphson iteration method is able to provide the failure and post-peak behaviour without any convergence issue. The results of the analyses performed adopting this latter constitutive law for different values of maximum shear stress are compared with those obtained with the Coulomb constitutive law adopting the Arc-Length method in Figure 6. As a displacement-controlled analysis is performed, the snap-back caused by the propagation of the cracking along the vertical interface cannot be captured.

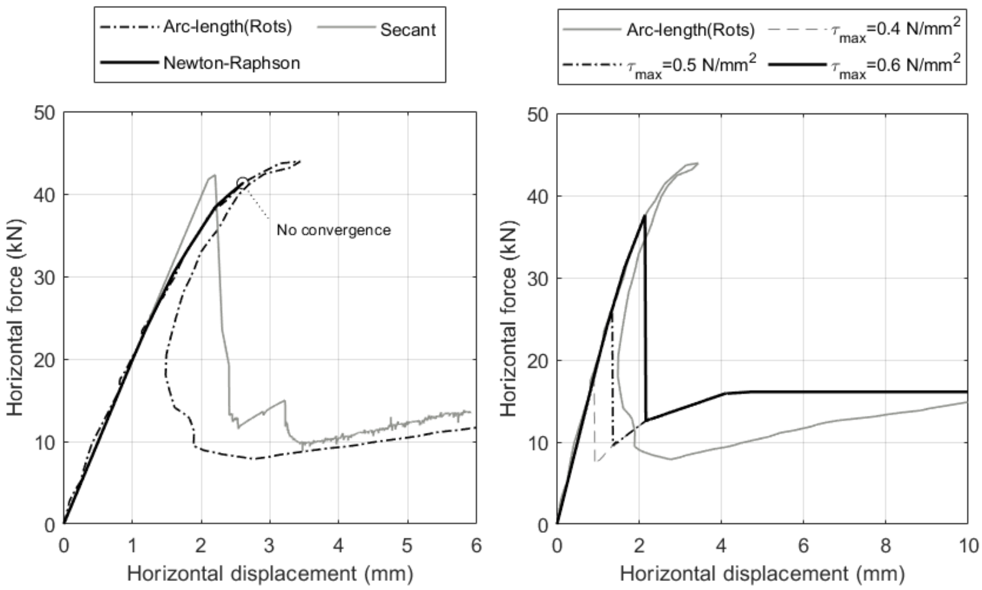


Figure 6: Capacity curves: Coulomb Friction Model [left] vs Nonlinear Elastic Model [right]

At the structural level, the nonlinear behaviour of the vertical glued connection between the pier and the transversal wall has been included in the three models (TSCM-Shell, TSCM-Solid and EMM-Shell) of the two-storey building described in the previous section. A vertical and a horizontal interfaces were introduced, as illustrated in Figure 7. The first represents the vertical glued connection. The rigid horizontal interface at the top of the pier was used to separate the node at the top of the interface.

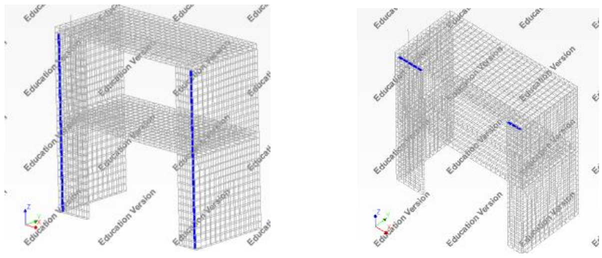


Figure 7: Vertical Interface with nonlinear constitutive law [left] and Rigid Horizontal Interface [right]

First, the connections have been modelled with a vertical interface characterized by the Coulomb friction failure criterion. Using the friction angle ϕ and cohesion of the element pier/wall, no failure of the vertical interface occurs and the results are equal to those obtained for the model without a vertical interface. Then, a sensitivity study was performed by varying the friction angle, to try to capture the failure of the joint. The capacity curves obtained for these variations are illustrated in Figure 8. The results of the sensitivity study show that modelling the vertical interface with Coulomb friction failure criterion leads to instability of the analysis and divergence occurs for small variations of the friction angle. Therefore, it can be stated that this model is not sufficiently robust and it is necessary to adopt a more stable constitutive law for the vertical interface.

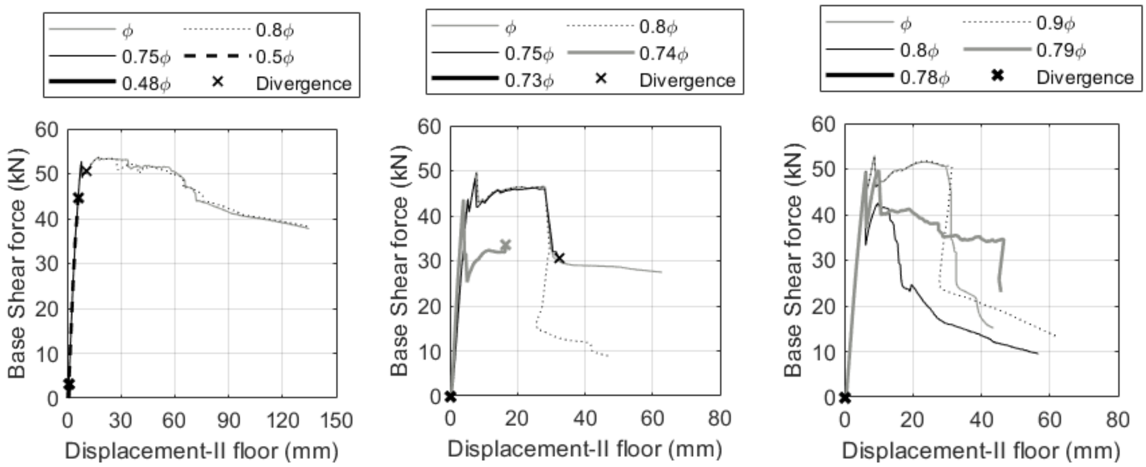


Figure 8: Capacity curves varying the friction for EMM-Shell, TSCM-Shell and TSCM-Solid model, with Coulomb friction model for the vertical connection constitutive law

Subsequently, the nonlinear elastic constitutive law was adopted for the interfaces and introduced in the same three models of the masonry assemblage. A sensitivity study of the influence of maximum and residual shear stress has been performed. The results of this modelling are significantly more stable than those obtained considering a Coulomb friction failure. Figure 9 illustrates the capacity curve of the structure for three different values of the maximum shear stress (τ_{max}=0.4, 0.5, 0.6 N/mm^2). The global seismic response does not change for larger values of the maximum shear stress. With the decreasing of this value, it is possible to observe that the sudden decrease of the capacity curve occurs for lower values of horizontal displacement. Therefore, the occurrence of failure mechanisms, as the diagonal cracking, depends on the strength of the vertical interface.

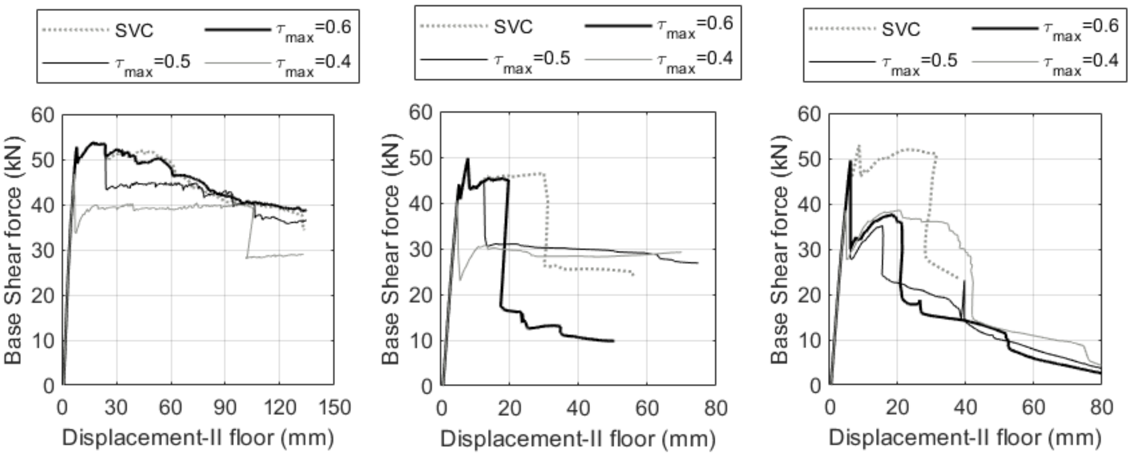


Figure 9: Capacity curves varying the maximum shear stress of EMM-Shell, TSCM-Shell and TSCM-Solid model, with Nonlinear Elastic Model for the vertical connection constitutive law

The sensitivity study varying the residual shear stress (τ_{res}=0.05, 0.025, 0.01 N/mm^2) is illustrated in Figure 9. The results show that the residual shear stress does not influence the peak value, but it rather governs the post-peak behaviour, since the residual capacity reduces at decreasing of this parameter, but the structure is then more ductile. Therefore, in other words, the sliding of the interface prevents the occurrence of further brittle failure, which corresponds to the additional cracking of the wide pier.

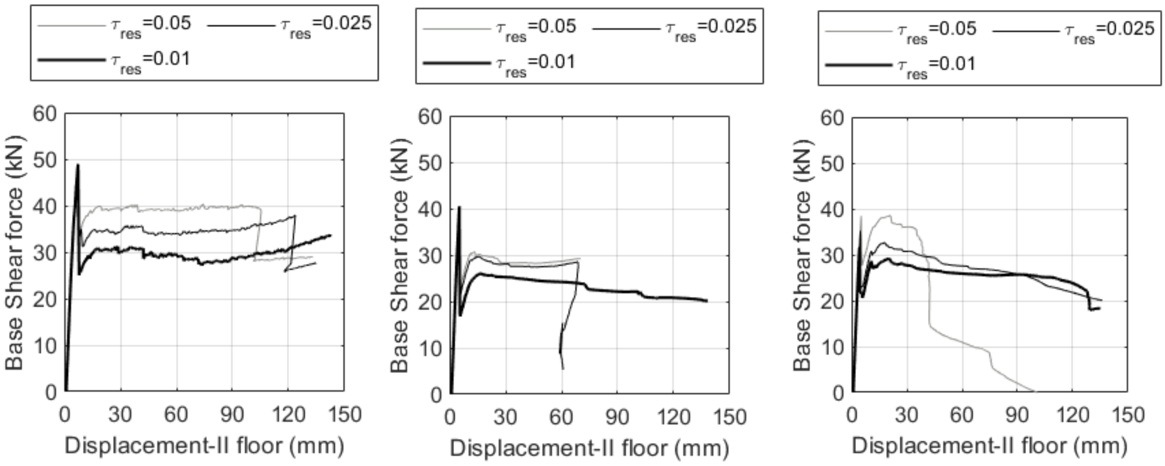


Figure 10: Capacity curves varying the residual shear stress of EMM-Shell, TSCM-Shell and TSCM-Solid model, with Nonlinear Elastic Model for the vertical connection constitutive law

4 CONCLUSIONS

The failure of the vertical glued connections between the wall and pier of the typical Dutch masonry buildings built after the 1980s may reduce the seismic capacity of the entire structure.

The behaviour of such vertical connections is analyzed first at the structural element level, by simulating a wall-pier system, and then at a structural level, by considering a full-scale two-storey building tested at TU Delft. This work shows that the choice of constitutive law is the most critical aspect of the modelling of these structures. The Coulomb-friction criterion is arguably the most representative constitutive law for the real behaviour of the connection, but its use leads to the numerical instability of the solution after the brittle failure of the vertical connection. An alternative modelling is then proposed for the connection by adopting the nonlinear elastic constitutive law defined by the relative displacement-shear stress diagram. In contrast to the Coulomb Friction model, this simplified constitutive law decouples the behaviour in the normal and tangential direction, and this leads to an improvement of the robustness of the model and the stability of the analysis. However, this law requires the calibration of the frictional parameters. Besides, it assumes a constant shear capacity along the height of the connection. When the vertical connection is considered in a complex model, as for example for a real building, the calibration of the parameter in a simpler model, as a pier-main wall model, is suggested. The limitations of this model suggest further investigation to define stable interface elements that may be able to properly represent the evolution of the Coulomb-friction behaviour.

The results of the sensitivity study show that the value of the maximum shear stress governs the occurrence of the shear failure of the interface, whereas the residual shear stress influences the post-peak behaviour. At the structural level, the shear failure of the vertical connection reduces the capacity of the building.

Furthermore, models with different types of elements or constitutive laws are used to assess the seismic capacity of the masonry buildings. The results show that the use of the shell elements or solid elements does not significantly affect the results. On the contrary, different constitutive laws, such as the Engineering Masonry Model or Total Strain Cracked Model, may determine different failure modes of the structure. In particular, when the Total Strain Crack Model is used, the collapse of the structure occurs for smaller lateral deformations.

Acknowledgements. This work has been developed as a jointed research cooperation between Sapienza University of Rome and Delft University of Technology, partly funded by the Sapienza thesis abroad scholarship program, herein gratefully acknowledged.

REFERENCES

[1] Rots, J. *Structural Masonry: An Experimental/Numerical Basis for Practical Design Rules*. A.A Balkema (1997).
[2] Raijmakers, T. M. J. and Van der Pluijm, R. Stability of the calcium silicate piers. *TNOBouw Report BI-91-0219* (1992).
[3] Esposito, R., Terwel, K., Ravenshorst, G., Schipper, R., Messali, F., and Rots, J. Cyclic pushover test on an unreinforced masonry structure reseambling a typical Dutch terraced house. In Proceedings of the *16th World Conference on Earthquake Engineering,*

Santiago, Chile (2017), pp. 1-12.
[4] Esposito, R., Jafari, S., Ravenshorst, G. J. P., Schipper, H. R., and Rots, J. G. Influence of the behavior of calcium silicate brick and element masonry on the lateral capacity of structures. In *10th Australasian Masonry Conference,* Sydney, Australia (2018).
[5] Messali, F., Pari, M., Esposito, R., Rots, J.G., and Den Hertog, D. Blind predictions of a cyclic pushover test on a two-storey masonry assemblage: a comparative study. In Proceedings of the *16th European Conference on Earthquake Engineering*, Thessaloniki, Greece (2018).
[6] Jafari, S., Esposito, R., and Rots, J. G. *From Brick to Element*: *Investigating the Mechanical Properties of Calcium Silicate Masonry*. RILEM Bookseries, 18, (2019), p.p. 596–604.
[7] Ravenshorst G., Esposito R., Schipper R., Messali F., Tsouvalas A., Lourens E-M. and Rots, J. *Structural behaviour of a calcium silicate brick masonry assemblage: quasi-static cyclic pushover and dynamic identification test*. Delft University of Technology, Department of Structural Engineering. Version 5, 21 October 2016 (2016).
[8] Magenes, G., and Penna, A. Seismic Design and Assessment of Masonry Buildings in Europe: Recent Research and Code Development Issues. In Proceedings of *9th Australasian Masonry Conference,* Queenstown, New Zealand (2011).
[9] Calderini, C., Cattari, S., and Lagomarsino, S. In-Plane Strength of Unreinforced Masonry Piers. *Earthquake Engineering and Structural Dynamics* (2008), 38(2), p.p. 243–267.
[10] Sacco, E., Addessi, D., Sab, K., *New trends in mechanics of masonry*, Meccanica, (2018), 53 (7), p.p. 1565-1569.
[11] Lourenço, P.B. *Computations on historic masonry structures.* Prog. Struct. Engng Mater. (2002). 4:301–319.
[12] Lourenço, P.B. *Computational strategies for masonry structures*. PhD Thesis. Delft University of Technology (1996).
[13] Calderini, C., and Lagomarsino, S. Continuum Model for In-Plane Anisotropic Inelastic Behavior of Masonry. *Journal of Structural Engineering (ASCE)*, (2008), 134(2), p.p. 209–220.
[14] Addessi, D., Marfia, S., Sacco, E., Toti, J. Modeling approaches for masonry structures. *Open Civil Engineering Journal*, (2014), 8 (1), pp. 288-300
[15] DIANA FEA User's Manual. TNO Building and Construction Research. (2019).
[16] Mariani, V., Messali, F., Hendriks, M. and Rots, J. Numerical modelling and seismic analysis of Dutch masonry structural components and buildings. In Proceedings of the *16th World Conference on Earthquake Engineering*, Santiago, Chile (2017).