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The Influence of Brick Bond in a Brick Masonry Using Simplified Micro Modelling Approach

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Abstract

An analytical investigation is carried out to evaluate the influence due to bond types on mechanical properties of brick masonry. A case of in-plane loading has been considered to evaluate the mechanical behaviour. Likewise, in half-brick thick wall three bond types were chosen for comparison with conventional mortar joint strength. A simplified micro modeling approach has been used with the plasticity based constitutive models for numerical modelling. Three dimensional nonlinear simulations were carried with varying aspect ratio and pre-compression load in all the three bond types. A significant difference in non-linear behaviour was observed with variation in bond type.

Keywords

Simplified Micro Modelling Masonry, Surface-based cohesive behaviour, In-plane load, Brick bond, Header bond, English bond, Aspect ratio, Pre-compression

1. Introduction

Masonry materials are widely used for construction of structures and infrastructures in most of the underdeveloped and developing parts of the world. Due to its low material cost, easy availability and heat insulating properties, the abundance of masonry still remains even with the availability of newly introduced technologies performing much better structurally. Many factors govern the use of masonry structures affecting their orientation while construction. Among them the geological conditions and traditional values are major reasons. Likewise, the aesthetic point of view also controls the layout and design of masonry structures. Different brick layout designs creating a certain pattern for aesthetic view were used in historical brick masonry structures. This trend of using masonry creating a pleasant view could still be seen in Nepal.

The nonlinearity in masonry is associated with cracking and crushing of masonry which is heterogeneous and anisotropic by nature. Depending on the mechanical properties, boundary conditions, pre-compression load and aspect ratio of the wall, different failure mechanisms are observed. The brick and mortar interface is usually the weakest part of the masonry [1, 2]. The volume of mortar changes with brick bond types hence the orientation of the brick (unit) and mortar joint would cause a variation in mechanical property of the masonry walls. Likewise, the strength of masonry is also governed by the formation of tension and shear stress developed between the bonding. Mostly the study of the brickwork masonry has been mainly focused on the compression and compression/shear failure mechanisms, and few effort has been given for studying the tensile/shear failure. Hence in the case of masonry, the orientation of masonry units has not been considered as a major concern for designing. Taguchi (2015) [3] investigated the influence of bond types on mechanical properties of brick masonry experimentally by compression tests. English bond style and Flemish bond style were chosen and subjected to compression loads. The research work concluded with lower strength of Flemish bond type specimens and it is believed to be due to larger volume of mortar used in this type of bond.

There seems to be a lack of research work exploring the influence of brick bond type in mechanical behaviour of masonry walls. To pursue this goal, a numerical modelling approach has been used to investigate the influence of bond type in mechanical property of masonry wall. The in-plane loading condition has been considered for the study with the help of a simplified micro-modelling approach.

2. Simplified Micro Modelling

The modelling approach is implemented using the commercially available FE package, ABAQUS 2017. Abaqus/Standard was employed for solution algorithm, which is suitable for static and low-speed dynamic loadings with high accuracy in stress solutions.

2.1 Modelling Masonry Unit

For modelling the masonry units, 3D hexahedral shaped eight node linear brick elements (C3D8R) was used under control elements reduced integration and hour glass control. The brick unit has been modified to have the elastic response equivalent to the original masonry specimen as per suggested by Abdulla (2017) [4]. Drucker-Prager Plasticity [5] has been used to capture the crushing of masonry under compression for defining the plasticity of the brick unit. The use of experimental ultimate compressive strength value (σ_c) to define the compressive stress-strain curves required for the compressive behaviour of the numerical models was taken as suggested by Kaushik (2007) [6], Hence, the adjusted elastic modulus (E_{adj}) of the masonry unit was modified.

2.2 Modelling Masonry Interface

In ABAQUS [7], contact elements are used to simulate the interfaces. The contacts between adjacent masonry units are defined through surface to surface, general contact discretization method using zero thickness contact elements.

Normal and tangential behaviours have been defined to simulate the behaviour of interface. The normal behaviour allows the adjacent faces to remain connected and bear the compressive stress but fail in tension. The formation of separation and a cracking of the faces are formed when the normal stress becomes zero at the faces. Hard contact behaviour was defined for normal behaviour in between the adjacent surfaces of masonry units by the contact pressure-over closure relationship. Similarly, the tangential behaviour defines the shear behaviour of the adjacent faces which is defined with the use of friction coefficient. The yielding criteria in tangential sliding is governed by Mohr-Coulomb criteria. If stress exceeds the shear limit obtained from the criteria, two faces begin to slide on each other and shear stress will be retained constant depending on the normal stress value. By using this method, no any new parameters are added to the finite element solution but some displacement constraints are added to the problem.

2.2.1 Linear Behaviour of Interface

For defining the linear and fracture behaviour of joints due to traction separation behaviour between masonry units, a surface-based cohesive model has been employed along bed and head joints. This allows to obtain the structural response of masonry units. The equivalent stiffness for joint interface proposed by Lourenco (1997) [1] to define the elastic response of the joint interfaces has been implemented. To imitate interface behaviour in the elastic range, normal stiffness (K_{nn}) and shear stiffness(K_{ss} and K_{tt}) is defined. The normal stiffness is dependent on the Modulus of Elasticity of both masonry units (E_u) and mortar (E_m) and the thickness of mortar (h_m) as show in Equation 1. Likewise, the shear stiffness is defined based on the Modulus of Rigidity of masonry units (G_u) and mortar (G_m) and thickness of mortar (h_m) as shown in Eq. 2.

$$K_{nn} = \frac{E_u E_m}{h_m(E_u - E_m)} \tag{1}$$

$$K_{ss} \ \& \ K_{tt} = \frac{G_u G_m}{h_m(G_u - G_m)} \tag{2}$$

The elastic stiffness matrix $\mathbf{K}$ and separation vector δ corresponding the nominal traction vector $\mathbf{t}$ of the joint interfaces are connected by the relation in Equation 3.

$$t = \begin{Bmatrix} \mathrm{t_n} \\ \mathrm{t_s} \\ \mathrm{t_t} \end{Bmatrix} = \begin{bmatrix} K_{\mathrm{nn}} & 0 & 0 \\ 0 & K_{\mathrm{ss}} & 0 \\ 0 & 0 & K_{\mathrm{tt}} \end{bmatrix} \begin{Bmatrix} \delta_{\mathrm{n}} \\ \delta_{\mathrm{s}} \\ \delta_{\mathrm{t}} \end{Bmatrix} = \mathbf{K}\delta \tag{3}$$

2.2.2 Plastic Behaviour of Interface

The fracture energies and friction coefficient are important parameters in defining the plastic response of the masonry. The fracture energy defines the trend of non-linear response of the homogenized interface element after initial failure. The models, however, are not based on fracture mechanics. Plastic response of the joint interfaces has been achieved based on the tractions between masonry interfaces.

Damage initiation refers to the beginning of degradation of the cohesive response at a contact point. The process of degradation begins when the contact stresses or contact separations satisfy certain damage initiation criteria that is defined based on tractions between the masonry interfaces i.e. shear and tensile strength of the joints. The quadratic stress criterion, as shown in Equation. 4, is used to define damage initiation; this criterion is met when the quadratic stress ratios of masonry interfaces are equal to one. The effective prediction of damage initiation of joints related to mixed-mode loadings can be achieved with quadratic stress criteria. The mix mode loading in masonry joint interfaces are tensile stress in the normal direction (t_n) and shear stress in the two shear directions (t_s and t_t).

$$\left\{\frac{t_n}{t_n^o}\right\}^2 + \left\{\frac{t_s}{t_s^o}\right\}^2 + \left\{\frac{t_t}{t_t^o}\right\}^2 = 1 \tag{4}$$

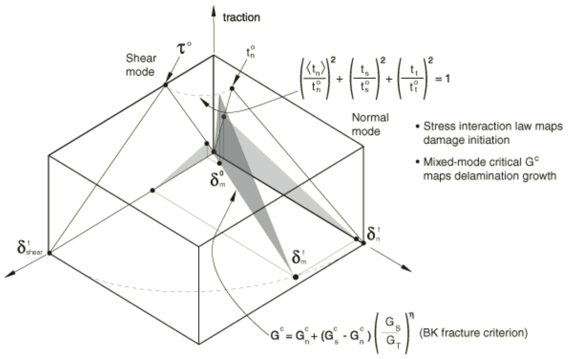


Figure 1: Mixed-mode response in cohesive interactions [7]

Damage evolution can be defined based on the energy that is dissipated as a result of the damage process, also called the fracture energy. The fracture energy specify the property of the cohesive interaction. A linear softening behaviour has been considered for the damage evolution as shown in Figure 2. ABAQUS ensures that the area under the linear damaged response is equal to the fracture energy.

The Benzeggagh-Kenane (BK) law [8] is used to obtain the critical mixed mode fracture energy (G^c) since it is the most suitable in the case when the critical fracture energies of both shear directions (mode II and mode III) are the same, which is the case in masonry joints.

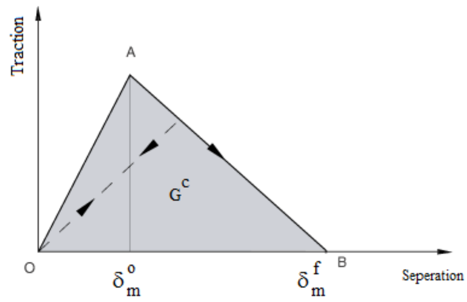


Figure 2: Linear damage evolution [7]

3. Verification of the model:

The experiment conducted by Raijmakers [9] on solid walls is an excellent sample to use for the verification of the numerical model. In the experiment, a specimen consisting of a pier with a width/height ratio of one (990 x 1000 mm ^{2}), with 18 layers of solid clay bricks (dimensions 204 x 98 x 50 mm) and 10 mm thick mortar (1:2:9, cement : lime : sand by volume) was considered. A vertical pre-compression force ”P” was applied on top of the walls before a monotonic horizontal load was provided under displacement control at top until failure as shown in Figure 3.

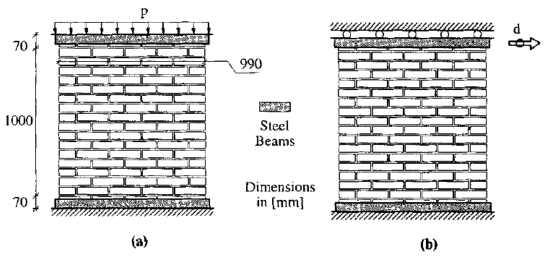
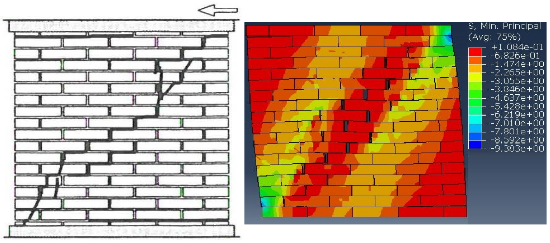


Figure 3: Loads for Solid Shear Walls: (a) Phase 1 - Vertical Loading; (b) Phase 2-Horizontal Loading [9]

For the verification, all three solid walls tested has been considered and a simplified micro modelling approach adopted as presented in Section 2 was used to make a numerical model. The parameters have been considered based on the previous research work on micro modelling, all parameters used in this validation study are summarized in tables 1, 2 and 3.

In the experiment, the failure mechanism of the masonry wall was initiated with a horizontal tension crack developing at the bottom and top of the wall. Then a diagonal shear crack with increase in lateral displacement appeared leading to the collapse. Simultaneous cracks in the bricks and masonry crushing in the compressed toes was observed. The diagonal compressive strut was forced to spread through both sides of the hole leading to the more distributed cracking pattern as shown in Figure 4.



(a) (b)

Figure 4: Comparison of failure modes of wall J4D and J5D: (a) Experimental failure patterns; (b) S $_{mm}$ (N/mm^2) with failure pattern from numerical model (scale factor = 20).

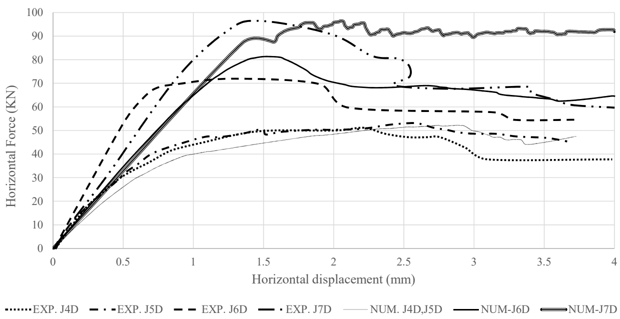
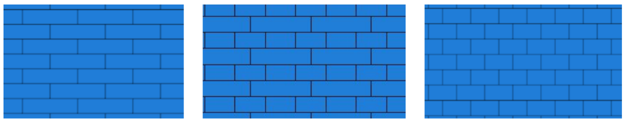


Figure 5: Comparison of pushover curve between experimental and numerical model

The damage initiation and formation of the numerical model obtained are in a close agreement with the experimental model as seen in Figure 4. Likewise, the horizontal force versus horizontal displacement curve of experimental and numerical model as show in Figure 5 also shows that the numerical model represents the experimental model closely. Hence, it can be concluded that the formulated micro model can simulate the masonry model in a complex loading condition with the high accuracy.

4. Brick Bond Modelling

In this section, the influence of physical properties of masonry wall due to brick bond in masonry wall is evaluated. Three kind of brick bonds has been considered as presented in figure 6. In type 1, stretcher bond type has been considered where the full length of the brick is used in bed joint. In type 2, alternative layer of full length and half-length of brick has been used in bed joint while in type 3, all the layers of masonry is built with half-length of brick unit.



(1) (2) (3)

Figure 6: Numerical Model of Masonry in ABAQUS: (1) Type 1, (2) Type 2 (3) Type 3

Material Properties: The mechanical properties for the modelling and brick units dimension has been considered to be same as in Section 3, of wall J4D and J5D. Hence, the study is limited to the moderated mortar bond strength.

Size of masonry assemblage : Aspect ratio of 0.5, 1 and 2 has been considered with height of the wall limited to 1m. Similarly, the thickness of the masonry walls have been considered constant with thickness of 100mm.

Loading and boundary condition: Two phase loading has been considered similar to the experimental test conducted by Raijmakers [9] (i.e, , a vertical pre-compression followed by lateral loading with vertical displacement restrained). Pre-compression of 0.05 and 0.1 times compressive strength of masonry was applied for each model.

5. Result

Numerical models were simulated as mentioned in Section 4. Horizontal loading versus the horizontal top displacement for each case is plotted as shown in Figure 7 ,8, 9.

From the numerical modelling, the result obtained clearly shows that the elastic response of the masonry wall is similar for all the bond type. However, the non-linear response varies with the bond types depending upon the combination of aspect ratio and pre-compression load. For the walls with H/L $\geq$ 1, the ultimate failure occurred due to diagonal cracking in both the pre-compression load. The type 1 wall had a slightly higher nonlinear range and shear capacity which was followed by type 2 and 3 respectively. However, for H/L =0.5 and pre-compression = 0.05 * σ_c, sliding failure occurs in type 1 and diagonal cracking followed by sliding in type 2 and 3. With increase in pre-compression load to 0.1 * σ_c in the wall with H/L=0.5, the ultimate failure occurred due to diagonal cracking in all three bond types.

Table 1: Elastic properties of constitutive materials and joint interfaces

S.N.	Elastic Properties	Masonry Constitutive	Solid Walls			Remarks
			J4D and J5D	J6D	J7D	
1.	Elasticity	Brick (E_u) (MPa)	16700	16700	16700	Lourenco (1997)
		Mortar (E_m) (MPa)	780	1030	780	
		Expanded Units (MPa)	4050	4655	4655	Abdulla (2017)
2.	Poisson Ratio	Brick / Mortar	0.15	0.15	0.15	Lourenco (1997)
3.	Joint Stiffness	K_{nn} (N/mm3)	82	110	82	
		K_{ss} , K_{tt} (N/mm3)	36	50	36	

Table 2: Nonlinear material properties for the joint interfaces

S.N.	Non-linear Interface Properties		Solid Walls			Remarks
			J4D and J5D	J6D	J7D	
1.	Tension	t_{nmax} (MPa)	0.25	0.16	0.16	Lourenco (1997)
		G_f^I (N/mm)	0.018	0.018	0.018	
2.	Shear	c (MPa)	1.4 *t_{nmax}	1.4*t_{nmax}	1.4* t_{nmax}	
		μ	0.75	0.75	0.75	
		G_f^{II} (N/mm)	0.125	0.05	0.05	
3.	Compression	σ_c (MPa)	10.5	11.5	11.5	

Table 3: Properties for the adjusted masonry units

Tension		Shear	
Tensile Strength (MPa)	G_f^I (N/mm)	Shear Strength (MPa)	G_f^{II} (N/mm)
2.0	0.08	2.8	0.5

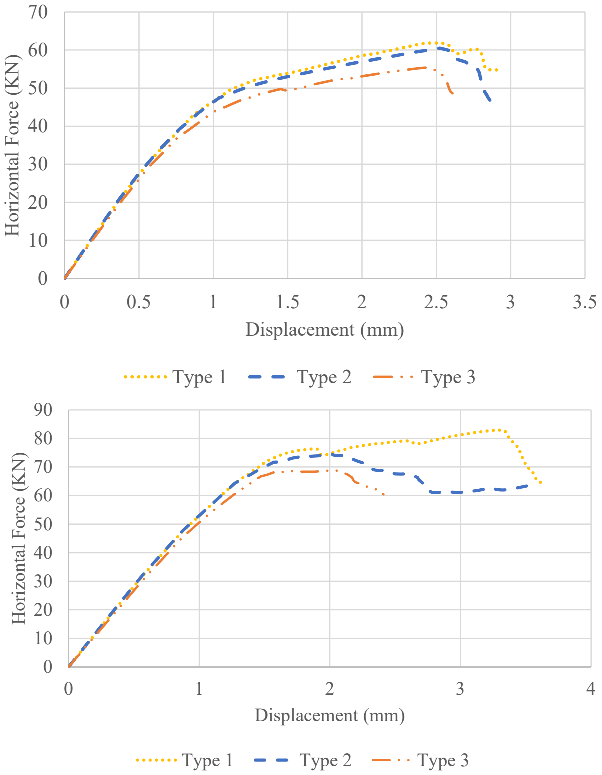


Figure 8: Force vs displacement for H/L=1; Pre-compression = a) 0.05*σ_c, b) 0.1*σ_c

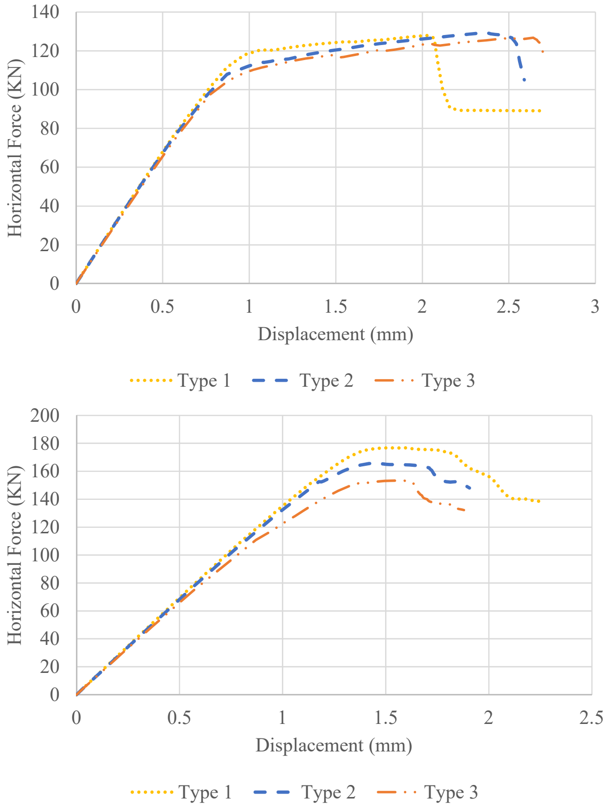
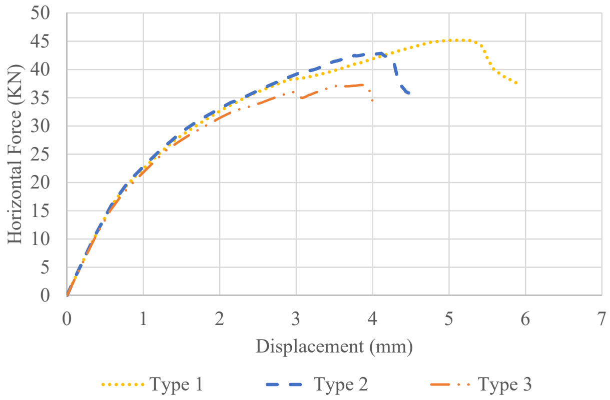


Figure 9: Force vs displacement for H/L=0.5; Pre-compression = a) 0.05*σ_c, b) 0.1*σ_c

As observed, the non-linear behaviour in masonry is governed by the failure in bed joints and head joints. Diagonal cracks in masonry walls developed under a combination of vertical and horizontal loads, when the tensile stresses exceeded the tensile strength of



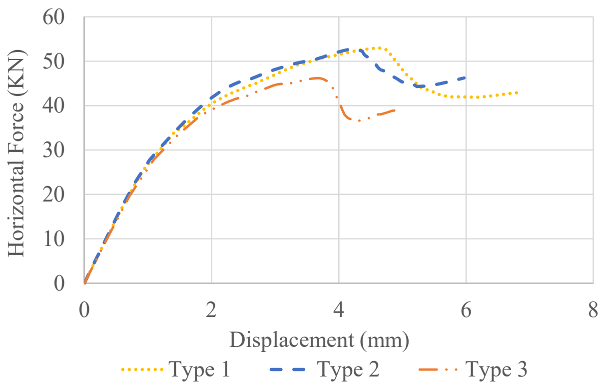


Figure 7: Force vs displacement for H/L=2; Pre-compression = a) $0.05\sigma_c$, b) $0.1^*\sigma_c$

the masonry material. For the aspect ratio (H/L) $\geq$ 1, the bond type with higher number of head joints reaches ultimate failure sooner as the diagonal cracks propagates quicker through the head joints. However, for the aspect ratio (H/L) <1, the pre-compression load has significant contribution in development of tensile stress in masonry wall causing the ultimate failure. For lower pre-compression load ($0.05\sigma_c$), the ultimate failure mechanism differs with bond type, as type 1 failed in shear sliding and the other two failed initially with diagonal failure followed by sliding shear. Each failure mechanism possesses its own strength and displacement characteristics [10]. Hence, type 1 bond fails in lower ultimate top displacement than other two. Similarly with increase in pre-compression load to $0.1\sigma_c$, increase in tensile stress in joint is seen which causes the ultimate failure of masonry regardless of bond type due to diagonal failure as in the case with aspect ratio (H/L) $\geq$ 1.

6. Conclusion

Three type of masonry bonds were considered to investigate their influence on mechanical properties of half brick thick masonry wall. The numerical investigation established that change in bond type affects the mechanical behaviour of masonry in in-plane loading condition. The walls with different bond types have same elastic behaviour but different non-linear behaviour. The combination of vertical and horizontal loads develops tensile stress which causes the variation in mechanical behaviour of masonry due to bond types. The bond type with more number of head joints seems to fail sooner compared to that with less number of head joints. Hence, a significant influence in mechanical behaviour of masonry due to bond type variation has been observed.

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