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IMPACT OF MODELING ARCHITECTURAL DETAILING FOR PREDICTING UNREINFORCED MASONRY RESPONSE TO SUBSIDENCE

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ABSTRACT:

More than one-fifth of the cost of major transport infrastructure projects relates to construction related damage prediction and mitigation. In tunnelling, recent attempts at using remote sensing as a less expensive alternative to traditional surveying for creating computational models of masonry buildings for better damage prediction raises fundamental questions as to the necessary quality of the input data, as there is a direct relationship between data quality and acquisition costs. Given the large quantity of potentially vulnerable buildings along a tunnel's route, features such as windows are typically considered only generally. To understand the implications of such choices and to better explore the viability of using remote sensing as input for computational models, 16 finite element models were devised to investigate the impact of window shape, brick orientation, window size, and the presence of lintels. Response was considered with respect to gravity loads and excavation-induced subsidence. Permutations of three common window shapes were modelled as representative of Georgian brick structures. The base model was benchmarked against large-scale

experimental work using a non-linear analysis. This study proves that a few simple assumptions can be used in reducing complexity of the building façades during reconstructing 3D building models for computation without causing major errors in structural response based on these models.

KEYWORDS: Masonry, Finite Element, Computational Modelling, Subsidence, Settlement, Excavation, Building Response, Brickwork, Windows, Terrestrial Laser Scanning, LiDAR

INTRODUCTION

Arguably, the political and financial success of the installation of major infrastructure projects may be tied to the level of damage reported during construction. Consequently, attempts are being made to convert remote sensing data into computational meshes to be able to generate first-order finite element method (FEM) meshes for individual structures [3,4,16,37] and entire sections of cities [17]. The goal of such work is to achieve reasonably good geometric representations of existing structures without incurring the usual time and costs required by traditional surveying. As the data quality of both laser scanning and photogrammetry can greatly differ depending upon the time expended for data capture, which is inherently linked to cost, decisions must be made as to the resources committed to a particular set of documentation efforts. This must be driven by the required quality of the output.

In current practice, terrestrial laser scanners (TLS) are increasingly being used to acquire elevation points of vertical surface of building façades in order to reconstruct the building models, because the technology can quickly collect data of high accuracy and high spatial data density. These data sets have potential as the basics to reconstruct models of existing buildings. Creating computational building models from the TLS involves three main steps:

data collection, data registration, and modelling [28]. For TLS data, the first two steps can be done within a proprietary software along with the TLS scanner. While for the modelling step, a geometric model is created manually, semi-automatically, or automatically depending upon the building complexity and functional requirements of the computational model. The building dimensions extracted from TLS data may differ from the actual values because of inaccurate points at spatial discontinuities (object edges) [36]. Also, assumptions are often made for which no systematic justification has not yet to be established, such a rectangular shape for each window and door of a façade [32]. Additionally, the building façade is generally treated as a homogeneous object, where some potentially important components (e.g. window lintels) are not modeled explicitly [7,32]. Clearly, these assumptions lead to reconstructed building models that differ from the actual ones, and finite element results based reconstructed building models may thus differ from expectations. The impacts of finite element analysis results from such simplification model have still not yet been explored. The influence of such assumptions must be investigated before adopting TLS for automatically generating solid models of the building façades for computational modelling.

Furthermore, when creating finite element meshes for predicting the performance of structures, the extent to which building features that may be considered primarily architectural in nature need to be modelled and the implications of their inclusion or exclusion is poorly defined. A significant example of this is brick patterning. Although many finite element programs have the capacity for element creation that could fully represent the position and orientation of every brick, this level of modelling is rarely done due to the resources that would be necessary both to create the mesh and to computationally process it. As such, the majority of architecturally-oriented building features are never modelled, but rigorous justification of this point is not well established.

To begin to quantify the ramifications of using simplified geometric models and omitting architectural components during structural analysis, this paper investigates the potential impact of such decisions using size, shape, and configuration of window openings and surrounding brick patterning of Georgian buildings, when subjected to excavation-induced settlement. Bearing walls with three types of window shapes (rectangular, arch, and wedge) along with several permutations, were modelled in order to investigate simplified assumptions and geometric errors generated when creating building models from the TLS. The global and local behaviours of the numerical models were considered.

BACKGROUND

A major concern when constructing underground structures in urban areas is the effect on adjacent buildings. Underground construction causes ground movements, which may negatively impact adjacent structures, especially unreinforced masonry buildings, because of their low tensile strength. For more than 50 years, researchers have developed and applied empirical methods both to predict the extent of ground movements [6,24,27,29] and to create safe limits for building movement under such conditions [8,9,11,13,31,34]. Recent advantages in FEM allow engineers to predict realistic behaviour of structures adjacent to an underground infrastructure project [10,30]. However, for a tunnelling project, hundreds of building structures must be assessed. The cost and time required for acquiring their geometric models by a traditional surveying is not generally feasible. However, recent efforts were achieved in generating a fully automated pipeline to convert remote sensing data into FEM meshes [17,21]. Using such data is not necessarily simple as laser scanning and photogrammetry are by their nature pixelized. So, their recording of objects consists of discrete points and not a continuum, such as a defined surface.

Irrespective of data input, building façade models are often treated as homogenous components [7,17,21,32]. This can be problematic as the complexity of analyzing masonry structures relates to the composition of units and mortar joints, as well as the presence of architectural details. As part of this, the importance of window position and the opening ratio (area of window/total possible wall area of solid wall) have been recognized as influential factors on building response. Asteris [5] used a FEM model of a one-story, one-bay frame to investigate the impact on stiffness of infilled frames by changing the opening ratio (OR) from 0% to 25% and by locating windows in various positions with respect to the compressed diagonal (beneath, upon, and above). Openings on the compressed diagonal reduced lateral stiffness 13%. Similarly, results by Mo et al. [26] on 65 full-scale masonry walls supported by a concrete beam, with variously positioned openings showed that when the masonry walls were subjected to static, distributed, vertical loads applied atop the walls, the openings exacerbated wall deformations, with patterns of differing principal cracks manifested around the top of the openings, as a function of specific combinations of bending, shearing, and local compression failures.

To help further understand the complexity of the situation, Achyutha et al. [1] conducted an elastic analysis of one- and two-story in-filled frames subjected to lateral loads with varied door and window positions. To represent stiff, medium, and flexible frames, 3 column cross-sections (400x400mm, 200x200mm, and 200x75mm) were used, and contact conditions were varied between the frame and infill. As expected, lateral stiffness decreased as the percentage of openings increased. The nature, magnitude, and distribution of principal stresses were also influenced by various opening types and positions, as well as the extent of contact between the masonry panels and frame. Where no contact between the frame and infill existed, tensile stress around windows was largely dissipated (being only slightly stiffer than bare opening cases). Shariq et al. [33] investigated many of the same issues through finite element analysis

by considering the seismic response of only one-room, within a single-storey masonry building, by differing the aspect ratios (ratio of length/width of the wall) and openings in the walls. As ORs increased, stresses around openings also increased, as well as the maximum principal tensile stress and maximum shear stresses. Similar work was done using discrete element analysis by Son and Cording [35] to evaluate equivalent bending stiffness and shearing stiffness of a masonry building taking into account both the anisotropy of the masonry units and the OR. Their parametric study indicated that the ORs and brick/mortar interface stiffness had more influence on a wall's shear stiffness than its bending stiffness, and that shear stiffness was critical to predicting the onset of cracking in masonry buildings adjacent to excavations [35].

To assess damage levels of masonry buildings near the construction of Madrid's Subway line 7, Melis and Rodriguez Ortiz [25] modelled adjacent buildings by a simplified beam model with elasto-plastic behaviour. The beam dimensions were based on the overall geometries of each building. The impact on stiffness of the openings (windows and doors) was evaluated by use of equivalent beam theory, in which the Young's modulus of the equivalent beam was re-defined based on the overall building stiffness including the openings. Reduction factors to account for the presence of the openings were suggested, as a function of the building's length-to-height ratio. Subsequently, Pickhaver [30] subjected masonry building facades with various opening positions and percentages, to both sagging and hogging within a tunnel-induced ground trough. In general, façades subjected to sagging deformation cracked diagonally, whereas façades subjected to hogging exhibited mainly vertical cracks. In this case, increasing the OR had little effect on the global façade response or maximum damage level, although crack pattern and local damage concentrations differed.

Currently, some commonly used criteria to anticipate damage levels of masonry buildings subjected to excavation/tunnel-induced ground movements are based on the concept of a building as a simple, linear-elastic, deep beam that is homogeneous and isotropic [9,11-13]. These criteria were established based on simple models, in which the presence of architectural details related to openings and lintels, as well as floors and roof diaphragms, were not considered. To better assess the viability of these approaches and to determine the level of detail needed when using remote sensing data, this paper investigated numerically the influence of window shapes, brick patterning, opening ratios, and the presence of lintels on both general and local response of the masonry buildings subjected to adjacent, excavation-induced subsidence.

SCOPE AND METHODOLOGY

In this study, three window shapes (rectangular, arch, and wedge) were modelled (Figure 1), along with several permutations, common to Irish Georgian architecture (1720-1840). For which, these different arrangements of brick or appearance of lintels along the top courses of the windows were considered representative of historic brickwork. The rectangular windows with lintels and without lintels (but with different arrangements of brick layers atop the windows) were considered to investigate an impact of treating façade as homogeneous. Additionally, the arch and wedge windows in the building were slightly larger than the rectangular window in order to examine influence of using a rectangular shape to fit for other styles of the windows. The original, overall geometry and window positions of a baseline mesh emulated an experimental model (as described below). Subsequent window shapes were placed to align with the bottom edge of the original window positions. The lengths of the window sides were held constant. Thus, where there was some form of an arch, the window size necessarily increased slightly. The building model having the rectangular

windows with the lintels is a numerical model of the large scale experimental test, while other models are based on this model by replacing the window styles.

Given the weak, out-of-plane connection details of most historical masonry and the higher vulnerability to horizontal strain when the long, load-bearing walls are oriented perpendicular to soil displacement, two-dimensional (or plane stress) analysis is often employed when evaluating a structure's vulnerability to tunnel or adjacent-excavation induced ground displacements (e.g. [18]). The approach has been considered a reasonable compromise between accuracy and resource allocation compared to the high computational expense and complexity of three-dimensional (3D) models. Such an approach has been adopted herein.

A total of 16 finite element analyses were conducted on two-dimensional (2D) masonry walls with semi-continuous footings (shallow and deep). A detailed micro-model was used to represent the buildings, where mortar and bricks were modelled separately, and an assumption of a perfectly bonded interface between the two materials was made (as opposed to having an interface element as well); in such a case, the failure of the mortar joint also reflects the failure of the brick-mortar interface. Non-linear material models were employed for both brick units and mortar, along with a smeared crack model to predict the onset of cracking.

The baseline FEM model was verified against previous 1/10th scale experimental work (Figure 2), where pairs of two-story high, unreinforced masonry walls were subjected to adjacent excavation perpendicular to the plane of the walls [18,22]. In that study, each wall was a single wythe thick of brick and had six windows per story. Each was 1.830m long, 0.610m high, 0.30m thick, and set 0.61m from each other (Figure 2). The mechanical properties of the masonry components were derived from laboratory tests of these

strength/stiffness scaled materials (Table 1). The excavation system was a sheetpile wall with three layers of tie-backs in a kiln-dried, poorly-graded sand, which was selected to match previous large-scale work. Full excavation and post-tensioning cycles were conducted, with data collected at each stage. Kinematic, as well as geometric, similitude were upheld (Figure 2); see building details in Figure 3.

The experimental walls consisted of individual bricks laid in mortar. The bricks were the smallest, commercially available, extruded units and were intentionally under-fired to achieve the required, scaled strength and stiffness properties. The mortar was of lime putty, with additives similarly designed to meet the scaling requirements (Table 1). The two tests (Test 4 and Test 5) reported herein for verification were part of a six-test program reported elsewhere [18,19].

The scale-model Tests 4 and 5 each included two unreinforced brick walls (named West and East). These were subjected to different loads but were otherwise identical. In the study presented herein, two of the four model walls were considered for verification: one under light loads with shallow footings – Test 4 East (T4E) and one with heavier loads and deeper footings – Test 5 West (T5W), as summarized in Table 2. Wall and ground displacements were recorded at multiple stages of the excavation process with dial gauges and linear voltage displacement transducers at 16 points on each model wall and throughout the soil mass and across the soil surface. At the design grade (corresponding to a scaled excavation depth of 1.22m in Test 4 and 1.09m in Test 5), the cumulative displacements occurring throughout the excavation process were recorded. Thus, this step was used for benchmarking. The experimental model wall displacements caused by adjacent excavation were manually imposed on the bottom of the wall models in the computational models (Table 3). Good correlations between the experimental results and expected empirical damage levels are

reported in detail elsewhere [20]. A full discussion of scaling and further description of the testing is provided in the following references [18-20]. Given the complexity of masonry modelling, the authors felt it was incumbent to employ a model for this study that had been verified under relevant loading conditions.

Geometric models

The baseline micro-model (MM1-1) was in part intended to verify the numerical model, in which lintels and the contact between the lintels and the building were modelled. The micro-model MM1-2 was similar to MM1-1 with enlarged windows, in which the OR increased from 18.9% to 21.4% (Table 4 and Figure 4a). None of the other models included lintels, in accordance with historical practices. Windows for all MM1, MM4, and MM5 models were rectangular, with the brick orientation atop MM4 vertically oriented and MM5 diagonally (Figure 4d-e and Table 4). Models MM2-1 and MM2-2 had arches (Figure 4b and Table 4), and MM3-1 and MM3-2 models had wedges (Figure 4c and Table 4), respectively. Wall apertures ranged from 18.9% in the base case (MM1-1), up to 21.5% in the model MM2-2 (Table 4).

NUMERICAL MODELING

As masonry behaviour is a function of mortar joints, bricks, and their interface, many modeling strategies have been proposed including detailed micro-modeling, simplified micro-modeling, and macro-modeling [23], from which only the detailed micro-modeling strategy uses continuous elements to represent bricks and mortars separately. Although separate treatment can give better results, it requires significantly more computational effort, because of the large number of elements needed (e.g. [23]). Based on previous research, only a detailed micro-model with perfect bonding between brick and mortar was shown to have

sufficient correlation to experimental work to justify adoption [38] for the problem being studied herein.

In an attempt to obtain realistic responses of masonry buildings, a non-linear analysis was adopted using ANSYS®, V11.0, which is based on the standard smeared crack model implemented into the 3D element solid65 [2]. This element has 8 nodes associated with 3 degrees of freedom, isotropic behaviour, and 2x2x2 integration points and is often used for modelling concrete. The uncracked material is modelled with an elastic-perfectly plastic law by Drucker-Prager yield criterion (Figure 5) [15]. As failure was not expected through other elements (i.e. lintels or footings), the solid45 element was used for these. This element is similar to the solid65 element but does not support prediction of cracking and crushing. In the physical experiment, the wall and lintels were not perfectly bonded. For modelling this interface, elements target170 associated with conta173 located on surface of 3D solid elements (i.e. solid45 and solid 65) were employed as interface elements between the wall and the lintels. Isotropic friction was applied for modelling this interface, with a coefficient of friction equal to 0.7.

These elements were selected as the most appropriate in the ANSYS library for modelling, the brittle masonry material. Specifically, the elements were able to accommodate the material constitutive law used for modelling crushing and cracking in masonry. This was the William-Warnke failure criterion [39], however, crushing was not considered, as the walls would be beyond a serviceability limit. The William-Warnke failure criterion was defined by five input strength parameters: ultimate tensile strength (f_t); ultimate compressive strength (f_c) [in which the stress space of the failure surface must be considered as $0 \leq \theta \leq 60^\circ$]; ultimate biaxial compressive strength (f_{cb}); ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state (f_1); and ultimate compressive

strength for a state of uniaxial compression superimposed on hydrostatic stress state (f_2). Of the five required parameters, two input constants (f_t and f_c) were adopted to specify the failure surface, and three were selected as program defaults: $f_{cb}=1.2f_c$, $f_1=1.45f_c$, and $f_2=1.725f_c$ [2]. Cracks at the integration points of each element were represented through the modification of the stress-strain relationship. A crack occurs at the principal weakness plane, if the principal stress becomes higher than the tensile strength of a material component. For consideration of subsequent loads that induce sliding across a crack plane, shear transfer coefficients were introduced: β_t for open cracks and β_c for re-closed cracks. Values of 0.2 and 0.8 were assumed, respectively, and the failure criterion for multi-axial stress state is given in Equation 1 [2]

$$\frac{F}{f_c} - S \geq 0 \quad (1)$$

where F is a function of the principal stress state, and S is a failure surface expressed in terms of principal stress and five input parameters f_t , f_c , f_{cb} , f_1 and f_2 uniaxial crushing strength. If equation 1 is satisfied, then the material will crush or crack.

As abovementioned, the Drucker-Prager yield criterion with a non-associated flow rule and no hardening rule was implemented for representing plastic behaviour (Equation 2)

$$f = \alpha I_1 + \sqrt{J_2} - k \quad (2)$$

where I_1 and J_2 are the first invariant stress and the second deviatoric stress invariant, respectively, and α and k are defined as Equations 3 and 4, respectively

$$\alpha = \frac{2 \sin \phi_f}{\sqrt{3}(3 - \sin \phi_f)} \quad (3)$$

$$k = \frac{6c \cos \phi_f}{\sqrt{3}(3 - \sin \phi_f)} \quad (4)$$

where ϕ_f and c are, respectively, the friction angle and internal cohesion.

According to Zucchini and Lourenco [40], the internal cohesion can be determined through the uniaxial experimental yield stress in compression, σ_c

$$c = \frac{1 - \sin \phi_f}{2 \cos \phi_f} \sigma_c \quad (5)$$

Additionally, the Drucker-Prager yield criterion in ANSYS® is basically determined by two material parameters: the internal cohesion and the friction angle. The flow rule is determined similar to that of the hardening rule but with the dilatancy angle replaced by the friction angle, in order to evaluate the direction of plastic strain, which controls volumetric expansion of the material with plastic strain. In this study, the assumed dilatancy and friction angle values were respectively 5° and 35°, for both brick and mortar joint elements in all of the numerical models. This means that the flow rule is non-associated, thereby resulting in less volumetric expansion. From the mechanical parameters adopted for both mortar and brick, the yield surface was assumed to be immutable with progressive yielding. Hence, there was no hardening rule, and the materials were considered perfectly plastic. In summary, an uncracked material is modelled with an elastic-plastic law governed by the Drucker-Prager plasticity, and cracking material under tensile stress was dominated by William and Warnke failure criterion, which provides the tension cut-off of the Drucker-Prager yield surface (Figure 5).

Based on arrangement of bricks and mortar joints in the wall, a discretization scheme of the model is shown in Figure 6, in which each brick is modelled with three solid continuum elements that are interconnected with both bed and head mortar joints (Figure 6a-b). For the lintel, as well as regions around tops of windows in various models, components were discretized by sweeping the mesh from the bounding areas (Figure 6c-h). For the footings, a vertical, four-level discretization was applied. Illustrations of the finite element mesh are shown Figure 6, and the numbers of nodes and elements of each model are presented in Table 4.

A non-linear solution was undertaken via a full Newton-Raphson interactive solution algorithm with displacement convergence criterion, in which the stiffness matrix is updated with every iteration [2]. A converged solution was obtained once the square root of the sum of the squares of the displacements was less than or equal to 0.05 [2]. Given the tunnelling community's reliance on displacement-based (as opposed to crack-based) damage limits for ground movement limitation, use of an FEM approach was considered more relevant than a discrete element approach.

The structure was subjected to self-weight, external vertical loads, and excavation-induced displacements. Boundary conditions were assumed fixed at the bottom of the footings. Applied loads and self-weight for the numerical model were identical to the experimental model test by means of a two-step application. In step 1, gravity loads in the form of self-weight and external loads were applied. In step 2, displacements and stresses from the first step were kept, and recorded experimental displacements were applied along the bottom of the building. Self-weight was defined as the gravitational acceleration, density, and volume of each element. For step 2, a linear interpolation was made from the experimental response monitored at four discrete points along the wall. From this, six displacement outputs were

derived for multiple excavation stages. They were imposed at the bottom of the wall as constrained displacements on a series of nodes along the bottom of the walls (Table 3).

DISCUSSION OF RESULTS

Verification of the numerical model is shown in Figure 7 demonstrating generally good agreement for both sets of tests at the design grade excavation level. Thus, having established the efficacy of this model for predicting the in-plane response of masonry walls subjected to excavation-induced subsidence, the model was adopted to investigate issues of window detailing. There were three main topics of interest: (1) the impact of small changes in the OR of the wall; (2) the influence of the brick arrangement/orientation around the tops of the windows; and (3) the effect of the presence of a lintel. Each of these issues relate to the detail to which a structure is documented, the accuracy of the survey, and the subsequent decisions to what extent those features are computationally modelled.

Table 5 presents the results of the numerical study with respect to discrete displacements. Values in parentheses are from the subsidence-induced component. Graphical representations of these displacements for the top, front corner of the wall are shown in Figure 8, as normalized by the output of the MM1-1 models (rectangular window with lintels and smaller OR). In the lightly loaded model (Figure 8a), the differences ranged from 3.66% less to 2.09% more. In the heavily loaded model (Figure. 8b), the differences were from 0.74% less to 8.41% more. As shown in Figure 8, the lintel inclusive model (MM1) greatly constrained displacements. This was seen in comparison to the other two rectangular windows (the fan-shaped MM5 and the solider brick of MM4, with the slightly higher OR: 19.4% vs. 18.9%). The maximum displacements were nearly constant, despite a 2.6% increase in OR.

The average displacement across each bay is shown in Table 6 compared to the base case. The maximum displacements of the T5W models were generally 10% less than the T4E ones. This was as expected, as the T4E applied displacements were greater than those of T5W. The exception was the T5W models with lintels, where the heavier load concentrated stresses around the window tops (as will be discussed subsequently). Of interest in Table 6 was the range of differences between models across each of the three bays and between the horizontal and vertical displacements, thereby attesting to the complicated interaction of applied load, applied displacement, and window features (i.e. OR, brick arrangement, and lintel inclusion). The specific displacements showed an even greater difference when compared to MM1-1 (the rectangular window with a lintel with the smaller OR) as depicted in Figure 8. Specifically, there was as much as an -8.68% horizontal and +15.03% vertical difference for MM3-2 in T5W (the wedge-shaped window with the large OR), and -3.07% horizontal for MM2-2 (the arch-shaped window with the large OR) and 8.06% vertically for MM3-2 in T4E.

In the arched windows (MM2), the initial horizontal displacements were greater and decreased slightly with an increased OR in the lightly loaded test T4E and increased marginally in the heavily loaded test T5W. Vertical displacement with the small OR models began at different levels but trended similarly and largely proportionally. Under the lighter load and greater displacements of T4E, the wedge-shaped window (MM3) experienced a nearly proportional change in both maximum horizontal and vertical displacements, with an increase in OR. Under the heavier load (T5W), the horizontal movements were disproportionately exacerbated (approximately 3.5% increase in horizontal displacement for only a 0.8% increase in OR). This is believed to be a function of some of the bricks being aligned with the stress path from lower left to upper right – somewhat akin to what is happening in the fan-shaped window (MM5). So depending upon the weight of the building and the amount of the displacement, a small change in the OR may or may not make quite a

bit of difference, especially when computing tensile strain and the damage associated with it. Despite MM4 and MM5 both having rectangular openings with nearly the same ORs (18.9% vs. 19.2%), their respective behaviours differed. MM4 with the soldier course of brick moved more than MM5 (especially horizontally), while MM5 with the fanned bricks seemed to be influenced by the combination of OR and loading levels, as it trended differently in the two tests.

As a point of comparison, the solid wall (MM-6) gave less movement than other ones. The average horizontal and vertical displacements atop the wall were respectively 0.1% and -10.99% for T4E and 0.38% and -19.00% for T5W when MM6 was compared to MM1-1. This was as expected, as the wall was stiffer and exhibited more solid body rotation and less localized movement differences.

There are also highly local effects, as shown in Figure 9, which depicts displacements along the top of window B1; there is a data point for each corner of each brick. Window B1 is the opening closest to the source of ground displacements (Figure 2). Windows with smaller ORs (18.9%-19.4%) are shown in parts (a) and (b) and those with larger ORs (21.4%-21.5%) in parts (c) and (d). Figure 10 is similarly arranged for window T6 located in the top corner furthest from the excavation (Figure 2). The stress path through the window runs from the bottom left to the top right, where the window width is represented as “L” going from left to right (away from the excavation).

Figure 9 shows largely similar responses for the three windows in both tests, although the fan window (MM5) displaced slightly less under the lighter loading/greater displacement case of the window B1 in T4E. This trend differed for window T6 (Figure 10). There the window

with the lintel moved slightly less, and the disjunction in the displacement happened in the front quarter, which again largely aligned the stresses with the brick orientation.

Under the heavier loading (T5W), the lintel is more effective in constraining movement. In that test, the extra load causes fewer variations to emerge between the models. This trend is also seen in Figures 10a and 10b depicting behaviour of the top backmost window.

The fan shaped bricks (MM5) exhibited the most pronounced local effects, most easily seen the back quadrant in Figure 9a and 9b, where the projected stress line is travelling through the window in an alignment parallel to the brick orientation. In Figure 10, the localized response is more acute both for the fan-shaped bricks of MM5 (Figure 10a and b) and in the relatively large displacements in the window centres compared to the constraints at the edges. Nowhere is this clearer than in Figure 10d with models MM2-2 and MM3-2. This distinction in of local response differs from the global one previously observed by Pickhaver [30].

Local differences in response due to the lintel can be seen around the entirety of the window in the stress pattern. In Figure 11a, the maximum principal stress is in the upper left corner, at the edge of the lintel. As the opening grows, the maximum stress changes sides, but remains at the lintel's edge (Figure 11b). When the lintel is removed and replaced by a soldier course of bricks, the maximum stress migrates upwards from the window opening to atop the soldier course (Figure 11c). For all others, the maximum stress is actually in the bottom left corner (Figures 11d-h). Understanding such potential differences may better inform monitoring and intervention decisions, as well as modelling ones. The lintels also showed a maximum stress concentration of nearly an order of magnitude more, with only a small further increase in OR nearly doubling this value. In contrast, the other models increased maximum stress levels at a lower rate, especially the arch, which gained only about a 10% stress increase for a 1%

increase in OR, in contrast to the ones with lintels, where a 2.5% increase nearly doubled the maximum stress. The lintels also generally transferred the peak principal stress from the lower portion of the wall between the first two footings to the top of the back lower windows (Figures 12a-d vs. Figures 12e-p), as a function of the lintels providing an enhanced mechanism to transfer stress laterally through the wall. This is in part a function of the fact that the non-rectangular windows resulted in minimal material between the top course of the lower windows and the lower course of the top windows. This significantly increased the stress across the remaining material. This is especially evident in models shown in Figures 12f, 12h, 12j, and 12l. Key findings from this section are summarized in Table 7. As expected, in general, an increased OR typically elevated the peak stress level, as previously shown with a single window type by Shariq et al. [33]. Many of the other effects appeared to be interactions between the loading level and window shape.

Beyond the specific observations, the question remains as the needed accuracy for building modelling and ultimately building protection. Going back to the issue of the adjacent excavation/tunnelling scenario, three standard measures of damage prediction have been applied often by engineers in that community: (1) angular distortion, (2) tensile strain, and (3) damage levels, as will be described in detail below.

Based on the model wall's length to height ratio of 3 and a linear interpolation of angular distortion limits proposed by Cook [14] for uniformly distributed loads on homogeneous materials, a limit range of $1/726$ to $1/34$ was adopted. As shown in Table 8, all T4E models with openings performed the same, and in T5W, response was uniform within each bay, with the last bay showing a lower level of movement. In each case the window shape, OR, or lintel inclusion was not significant enough to change the categorization. A similar situation arises when applying tensile strain limits based on cracking strain as proposed by Boscardin and

Cording [11] (Table 9). There the cracking strain was computed by subtracting the elastic strain from a total principal strain in the cracking element, while the total principal strain was directly obtained from the FEM solutions. Despite the apparently wide range of results (18.09×10^{-3} – 1254.49×10^{-3}) spanning two orders of magnitude, all results for the models with openings fell within the “severe and very severe” category. Similar results are shown in Figure 13, where the respective percentages of the wall according to various damage categories are shown. Again, the models did not exhibit significant differences from each other. What this and the results of Tables 8 and 9 imply is that despite notable divergences, the identified parameters of lintel, window shape, brick arrangement, and small variabilities of opening ratios (up to 2.6%) have few practical implications for the tunnelling community given their continued reliance on fairly broad damage characterization methods; a 2.6% difference in opening ratio translates to around 20 mm difference in window size for the prototype buildings (around 2 mm for the window in the scale model with a scale factor of 10). Although this is now in a range of the possible accuracy of TLS, reconstructed 3D models may actually result in significantly larger errors in the base geometry depending upon data acquisition and processing strategies [36].

As such, there appears to be reasonable justification of the use of some laser scanners and possibly other forms of remote sensing for initial geometric model generation in this application. The one major caveat appears to be in the implications for the positioning of construction period monitoring efforts and any preceding condition assessment, as the peak stresses for walls with lintels greatly differed in position from those without.

CONCLUSIONS

A total of 16 finite element model masonry walls were subjected to excavation-induced settlements to investigate the impact of window features on building response. The goal was to determine the relative importance of modelling small-scale architectural features related to window composition for damage prediction in tunnelling and adjacent excavation. Results showed that a building's in-plane performance was most impacted by the presence or absence of lintels, which tended to concentrate stress and change the location of peak stress further from the point of highest displacement, as well as moving it from window bottoms to window tops. The presence of a lintel also made a disproportionately high increase in stress that manifested itself further back in the model, as a function of increases in the wall's opening ratio, despite reduced displacements, and the lintels' increased capacity to transfer the stress. Of the shapes tested, arched windows were most effective at stress distribution. However, localized stress distribution occurred where mortar joints were aligned with general stress paths. Despite all of these differences and a fairly wide range of responses, when the results were considered with respect to current limits employed by the tunnelling community (angular distortion, critical strain limits, and damage levels), only minimal differences were found. As such, there appears to be reasonable justification for use of remote sensing data as the basis for documenting geometry for the generation of computational models of existing structures for adjacent excavation and tunnelling damage predictions. Also, this study proves that a few simple assumptions can be used in reducing complexity of the building façades during reconstructing 3D building models for computation without causing major errors in structural response based on these models for this test case. However, advances in this area may profoundly change future expectations and resulting accuracy in pre-construction damage prediction for major infrastructure projects.

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Figure 2. Schematic 1/10th scale testing arrangement

Figure 3. Details of the experimental model masonry wall

a) MM1(-1 and -2): style 1 b) MM2(-1 and -2): style 2 c) MM3(-1 and -2): style 3 d) MM4: style 4 e) MM5: style 5

Figure 4. Schematic arrangements of mortar and bricks around windows

a) For mortar joints

b) For brick

Note: (+): Tension; (-): Compression

Figure 5. Drucker-Prager yield surface and William-Warnke failure criterion in the principal stress σ_I - σ_{II} space

a) Brick and mortar arrangement

b) Finite element scheme

Figure 6. Discretization scheme of the model and finite element mesh configuration around windows

Figure 7. Comparison of experimental vs. numerical masonry wall displacements of tests T4E and T5W measured along top of wall results at the design grade excavation level

Figure 8. Relative difference of displacements at a top front corner of the wall compared to MM1-1 at the design grade excavation level

Figure 9. Vertical displacements at top of window B1 in T4E and T5W at the design grade excavation level

Figure 10. Vertical displacements at top of window T6 in T4E and T5W at the design grade excavation level

Figure 11. Principal stress 1 around window T6 in the T4E at the design grade excavation level

a) MM1-1 for T4E

b) MM1-1 for T5W

c) MM1-2 for T4E

d) MM1-2 for T5W

e) MM2-1 for T4E

f) MM2-1 for T5W

g) MM2-2 for T4E

h) MM2-2 for T5W

i) MM3-1 for T4E

j) MM3-1 for T5W

Figure 12. Principal stress 1 in model at the design grade excavation level

Figure 13. Percentage of area of masonry wall damaged in T4E and T5W at the design grade excavation level