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Steps toward a probabilistic framework for tunnelling damage

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Risk Analyses and Techniques for Underground Structures

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1. Introduction

Globally, the high rates of urbanization over the past century have spurred unprecedented levels of tunnel construction. With each tunnel installation, there is a large affiliated risk for damage to aboveground structures, especially those of unreinforced masonry. Such damage (and the subsequent costs and litigation) occur, despite huge sums committed to construction monitoring and pre-tunnel mitigation. Arguably, damage still happens because the wide range of parameters and the extent of their variability are not sufficiently considered in the risk assessment process. To address these uncertainties, a probabilistic framework for the large-scale risk assessment of existing, unreinforced masonry buildings subjected to bored tunnelling is proposed by the Urban Modelling Group (UMG) at the University College Dublin (UCD). This paper summarizes the initial steps needed to achieve such a framework.

2. Background

In 1995 Burland (1995) proposed a 3-stage process for assessing risks related to tunnelling-induced settlement for large building stocks. In that, stage 1 relied on the generation of a greenfield settlement trough to establish whether a maximum slope (1/500) or a maximum settlement (10mm) of the ground surface is exceeded at the location of any building. Any building at such locations was to be considered in Stage 2. Stage 2 then considered each building as a simple deep beam whose foundations were assumed to follow the greenfield ground displacement profile. Subsequently, the maximum tensile strains were calculated, and an appropriate category of damage was assigned to the building. The goal of these two steps was to determine which buildings (if any) should be further considered for detailed analysis (Step 3).

The proposed detailed analysis of Stage 3 involved detailed considerations of tunnelling methods, structural continuity, foundation characteristics, building orientation, soil-structure interaction, and previous displacements (Burland, Standing et al. 2001). As part of this, the subsidence trough (shape and magnitude), time-dependent movement, protective measures, and damage level (from both subsidence and horizontal strains) were also considered, as well as the three-dimensional (3D) stiffness effects of the building undergoing displacement.

Early attempts at this were made by Pickhaver (2006), who extended the 2D deep beam model into 3D but did not incorporate window and door openings. Thus, the resulting stiffnesses were greatly overestimated, and stress concentrations around apertures (where damage usually initiates) could not be considered.

Schuster (2008) also tried to address the factors identified by Burland (1995) but in a very different way. In this case, the 2D arrangement was retained, but a framework for a fully probabilistic analysis for cut and cover tunnelling in soft clays was proposed based on a serviceability limit state. Using previously published case studies, damage levels were developed based on maximum crack widths, and a semi-empirical model was used to generate a vertical ground movement profile. Finally, a first order reliability analysis (which is probabilistic in nature) was conducted by applying the semi-empirical model. Subsequently, a finite element model was used for calibration of the variables in

the semi-empirical model, which enabled retention of the original closed-form solution.

While the influence of existing structures on subsidence troughs has been studied elsewhere, the approaches have generally been experimental (Laefer 2001, Son and Cording 2005, Son and Cording 2007, Son and Cording 2008, Laefer, Ceribasi et al. 2009, Laefer, Cording et al. 2010) to investigate the influence of building weight and bending and axial stiffness of an existing structure on tunnelling-induced ground movement. However, that research relied upon 224 deterministic, 2D models and did not incorporate a systematic or a random generation of testing scenarios. As part of the work, Law (2012) concluded that neglecting the building load can result in a non-conservative estimate of a subsidence trough, despite the numerical studies by Potts and Addenbrooke (1997) that indicated otherwise because of the stiffening contribution made by the building; the topic remains one for further investigation.

Giardina (2013) addressed the problem of building stock vulnerability with a very different strategy, which explored individual and small groups of buildings in 3D. The torsional building responses were considered through the development of a coupled, 3D model of the building, foundation, soil, and tunnel. While the research focused on a number of parameters that were expected to have significant impacts, the study was constrained by the numerous geotechnical assumptions that were made (e.g. use of a linear soil and omission of physical processes related to drained and undrained loading, consolidation, and ground water conditions). As Giardina (2013) did not use the concept of probability in this work, the relationship between the assumed coefficients and the analysed parameters were defined by weighting of parameters, and the interactions between the parameters were not considered explicitly.

In summary, despite many notable publications in this area, there has yet to be published a study proposing a comprehensive, probabilistically based risk assessment of existing buildings in 3D that can be effectively applied to a large group of buildings. Even the recent study by Clarke and Laefer, which involved the condition of the building and its perceived value in the community or the subsequent work by Clarke (2015) towards a fully-coupled model largely ignored the uncertainties related to the geological conditions and those of the construction processes.

3. Sources of Uncertainties for a coupled model

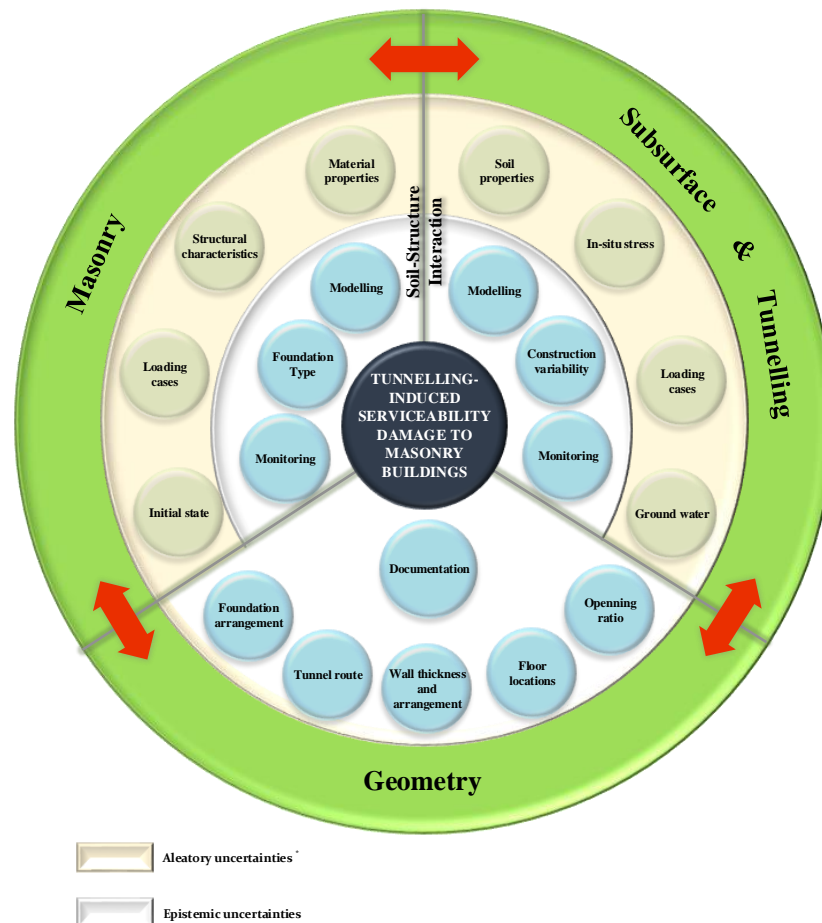
To devise a framework to predict tunnelling-induced, serviceability-level damage to unreinforced masonry buildings, all potential variables that propagate uncertainties must be identified. Excluding human errors and omissions, the uncertainties associated with an appropriate coupled numerical model (CNM) can be divided into two general categories: epistemic and aleatory.

Epistemic uncertainties represent those due to lack of knowledge, whereas aleatory uncertainties represent the natural randomness of a variable (Nadim 2007). Most of the uncertainties that are aleatory can be initially epistemic due to the lack of knowledge. Uncertainty quantification works towards reducing epistemic uncertainties to aleatoric uncertainties. Epistemic uncertainty can be reduced, and perhaps eliminated, through more extensive data collection, improved measurement methods, or refined calculation means (Nadim 2007).

In contrast, aleatory uncertainties cannot be reduced or eliminated (e.g. seasonal location of the ground water table or spatial variability of a soil parameter). Some aleatory uncertainties, such as soil properties are best defined as random variables described by their mean, standard deviation (or coefficient of variation), and probability distribution function. However, epistemic uncertainty such as measurement uncertainty is best described in terms of accuracy, bias (systematic error), and precision (random error) and can be evaluated from manufacturer-generated or other experimental data (Matthies 2007, Nadim 2007).

For a CNM, there are three categories in which such uncertainties exist: the tunnelling and its surrounding subsurface, the masonry materials, and the problem geometry. Each source can be divided into groups of uncertainties, as illustrated in Figure 1. For numerical modelling of uncertainties, the appropriateness of the results rely on the effectiveness and limitations of the applied approach. To evaluate this type of uncertainty, efforts are usually made to gain better

knowledge of the system, process, or mechanism.



* Aleatory uncertainties can be initially epistemic due to the lack of knowledge

Fig. 1 Potential variables involved in a framework for predicting tunnelling-induced serviceability damage to masonry buildings

To show the effect of numerical modelling uncertainty in a coupled modelling including damage assessment of a masonry structure, Moradabadi and Laefer (2014) compared four available approaches [micro-poly methods (MPM), distinct element method (DEM), discontinuity deformation analysis (DDA) and combined continuum-interface methods (CIM)] with finite element modelling (FEM). Figure 2 provides an overview of those methods, with respect to their ability to define the initial state, availability of input data, applicability of crack modelling, ease of model implementation, and the computing costs. The differences are shown qualitatively by the size of the marker, with the larger markers indicating beneficial attributes, and the smaller ones less beneficial characteristics. What is readily apparent is that standard FEM approaches are poorly suited to modelling damaged masonry, while DEM is extremely well-suited (at least under static and pseudo-static loading), and the other three methods are somewhere in between. The CIM is nearly as good as the DEM, except in the case of pre-deformed structures or when large deformations are expected. The DEM also has the advantage of being independent of mesh size. The MPM and DDA methods both had a wider range of limitations. Finally, although more applicable in explicit crack modelling than FEM or MPM, the complexity of DDA model generation, computational effort, and high input data requirements are all sources for concern. When properly applied, DDA does however perform well, especially for establishing a masonry structure's initial state of stress.

Arguably, to treat the epistemic and aleatory uncertainties of a coupled soil-structure model for assessing the serviceability damage of tunnelling-induced settlement of a masonry building, one must consider the best approach to calculate the damage (or, at least, to manage the limitations of the applied approach) and address the uncertain parameters that are estimated from a limited data set. As the risk of damage directly relies on the defined model parameters, the inclusion of stochastic models can be instrumental.

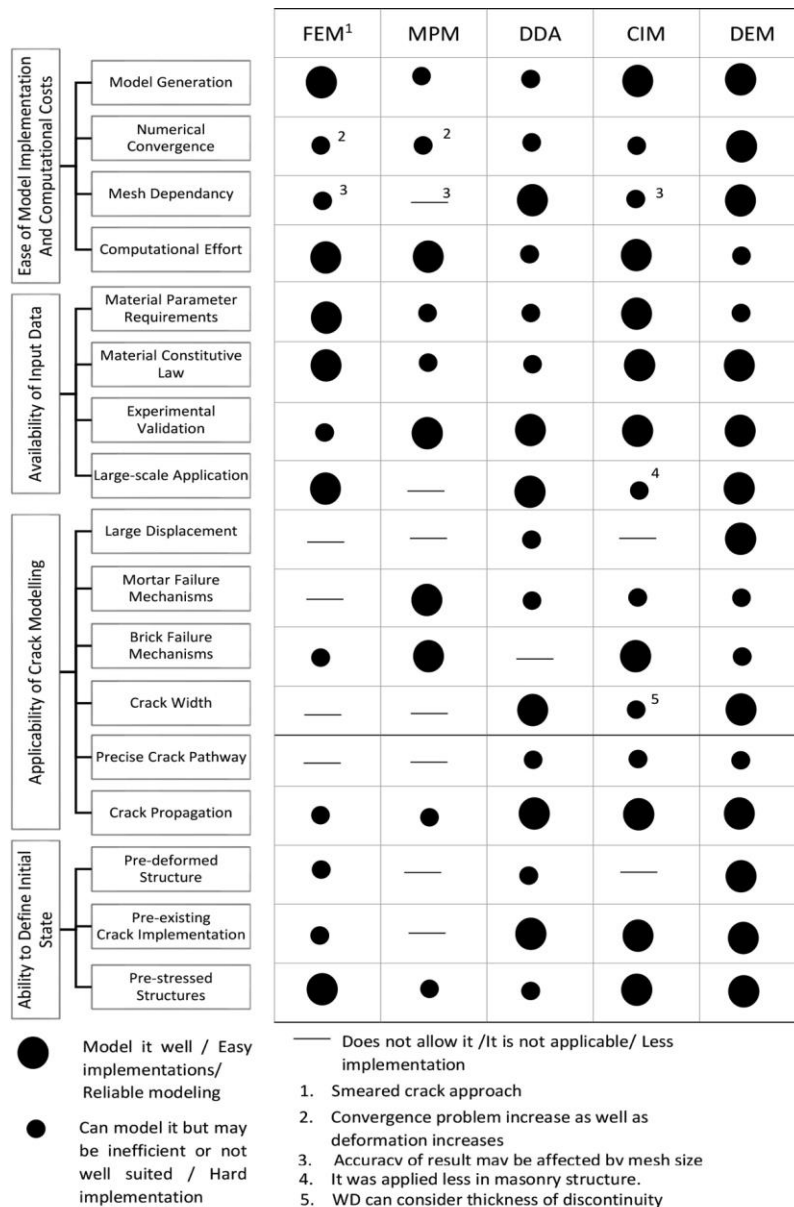


Fig. 2 Comparison of reviewed models versus FEM

4. Vulnerability analysis framework

The concept of performance-based design/assessment (PBD) is well-known in seismic design (FEMA 2012). PBD is based on risk methods that consider hazards, vulnerabilities, and consequences. The objective of PBD is to achieve a specified level of performance, as correlated to appropriate consequences, which may be measured in numerous ways (e.g. monetary loss and loss of life) [Figure 3].

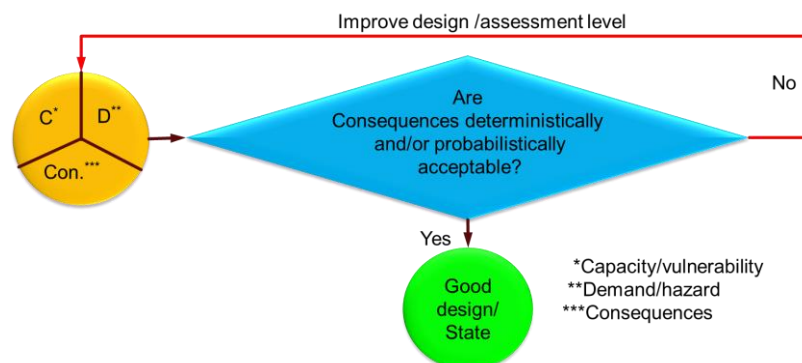


Fig. 3 Concept of performance-based design (PBD)

For the vulnerability analysis framework (VAF) proposed in this paper, the same concept was adapted to assess the performance of a masonry structure due to tunnelling-induced settlement. This considers different levels of tunnelling induced-settlement as identifiable hazard categories. The anticipated consequences and corresponding costs can then be calculated to describe the damage of the aboveground masonry structures (Figure 4-a). Given the extent of the hazard of the tunnelling (i.e. settlement magnitude), the probability function of the damage level occurrence for each consequence can be illustrated as a fragility curve (Figure 4-b). Subsequently, the damage level of the structure can be shown by a probability distribution function (Figure 4-c), and the consequence (risk) results from the multiplication of the cost of the damage and the damage level.

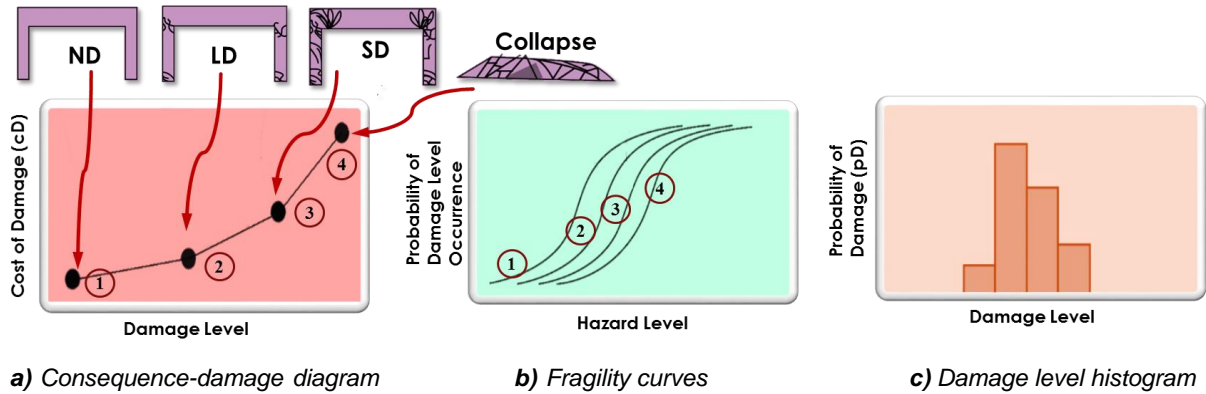


Fig.4 Computation of risk in vulnerability analysis (ND: No damage, LD: Low damage, SD: severe damage)

4.1 Performance limit function for settlement

To assess the system at a specified level of performance (Figure 3), failure should be defined in the VAF. In performance assessment, failure is classically defined as the conditions under which a pre-defined limit state is reached. A limit state function, which defines performance as either safe or failure, can be related to strength failure, serviceability failure, or anything else that describes unsatisfactory performance. Here, the performance limit function, f , is defined in equation 1 according to (Griffiths and Fenton 2007):

$$\begin{aligned}
 f(X) &\geq 0 \rightarrow \text{Safe} \\
 f(X) &< 0 \rightarrow \text{Failure} \\
 X &= [x_1, x_2, \dots, x_N]
 \end{aligned} \tag{1}$$

where X is the vector of model input, and N is the number of random variables, which come from predefined uncertainties. For a model with the estimated displacement/strain, δ_p , f can be translated as equation 2

$$f(X) = \delta_t - \delta_p(X) \tag{2}$$

where δ_t is a tolerable displacement/strain established by the designer/assessor, and $\delta_p(X)$ is the probabilistic distribution function (PDF) of the estimated displacements. If $\delta_p(X)$ is a parametric numerical element model, it is complex and has neither a closed form solution nor a known type of PDF. Thus, to generate a PDF or a cumulative distribution function of $\delta_p(X)$, a Latin Hyper Cube algorithm (Helton and Davis 2003) is proposed to simulate system uncertainty and to perform a reliability analysis as part of the CNM.

4.2 Performance criteria for damage level

To describe the tolerable strain and estimated strain in equation 2 and introduce the concept of a probabilistic damage level, reviewing the historical criteria for subsidence damage classification of masonry structure is helpful (Figure 5); for more information see the following references ((Burland and Wroth 1974, Burland and Wroth 1977, Boscardin and Cording 1989, Burland 1995).

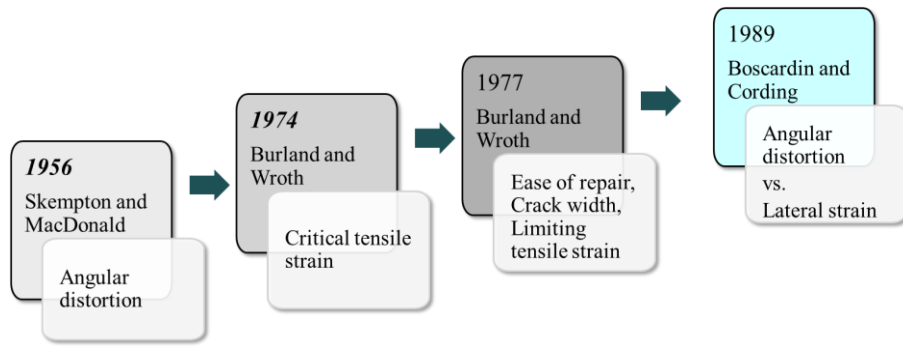


Fig. 5 Chronological development of damage criteria for masonry structure

The damage classification of a building due to settlement based on an angular distortion criterion was introduced first by Skempton and MacDonald (1956). Subsequently, Burland and Wroth (1974) discussed damage prediction by highlighting the difference between hogging and sagging deformation modes and by proposing a critical tensile strain based classification of masonry damage.

In 1975 and 1976 much of Europe was subject to severe droughts. As a consequence, many buildings on clay soils experienced damage. Based on that experience, (Burland and Wroth 1977) classified masonry building damage according to ease of repair. Considering the tensile strain as a serviceability parameter, limiting tensile strain was introduced, instead of the critical strain concept. Eventually, Boscardin and Cording (1989) added the influence of horizontal ground strain to the Burland and Wroth (1977) criteria. With this improvement, a range of values of limiting tensile strains could be assigned to the different damage categories.

Son and Cording (2005) then extended this concept through the applications of equations 3 and 4 relating to the calculation of principal strain, ε_p , at a specific point in a building:

$$\varepsilon_p = \varepsilon_1 \times (\cos(\theta_{\max}))^2 + \beta \times \sin(\theta_{\max}) \times \cos(\theta_{\max}) \quad (3)$$

$$\tan(2\theta_{\max}) = \beta / \varepsilon_1 \quad (4)$$

where β is the angular distortion as defined with figure 6, ε_1 is the lateral strain (i.e. horizontal strain), and θ_{\max} is the direction of crack formation measured from vertical plane (i.e. the angle of the plane on which principle strain ε_p acts).

When ε_p is calculated for a parametric model with the input vector of X , ε_p can be shown as $\varepsilon_p(X)$. By substituting $\varepsilon_p(X)$ with $\delta_p(X)$ and considering a tolerable limiting strain, ε_t , in Equation 2, the performance limit function transforms to Equation 5.

$$f(X) = \varepsilon_t - \varepsilon_1(X) \times (\cos(\theta_{\max}(X)))^2 + \beta(X) \times \sin(\theta_{\max}(X)) \times \cos(\theta_{\max}(X)) \quad (5)$$

$$\tan(2\theta_{\max}(X)) = \beta(X) / \varepsilon_1(X) \quad (6)$$

where ε_t can be classically determined by values of limiting tensile strain corresponding to the different damage categories (Table 1) defined by Boscardin and Cording (1989) or by the limiting principal strain as defined by Son and Cording (2005).

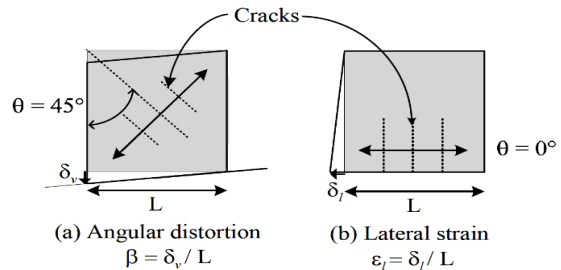


Fig. 6 State of strain at the distorted portion of a structure (after Son and Cording (2005))

Table 1-Relationship between category of damage and principal strain

Category of damage	Normal degree of severity	Limiting tensile strain (%) (Boscardin and Cording 1989)	Limiting principal Strain (%) (Son and Cording 2005)
0	Negligible	0-0.05	0-0.05
1	Very Slight	0.05-0.075	0.05-0.075
2	Slight	0.075-0.15	0.075-0.167
3	Moderate to severe	0.15-0.3	0.167-0.33
4 to 5	Severe to very severe damage	>0.3	>0.33

5. Applying the framework to a case study

To briefly show the application of the new framework for damage assessment of a masonry structure subjected to tunnelling, a 3D CNM built up by Clarke (2015) (Figure 7 and 8) was examined; for more information on the original case study see (Burland, Standing et al. 2001)

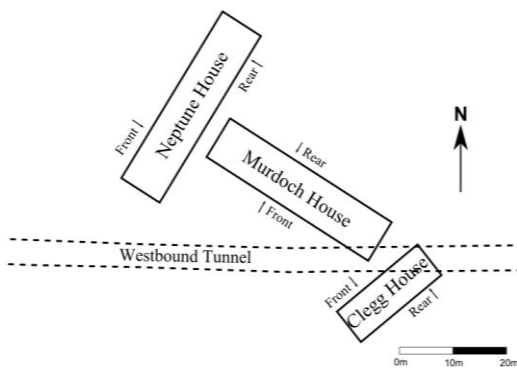


Fig. 7 Plan view showing Jubilee Line Extension case study

The original coupled model consisted of a group of three buildings: Neptune, Murdoch and Clegg Houses. All were located on Moodkee Street in London, UK, which underwent deformation as a result of the adjacent Jubilee Line Extension. Two parallel tunnels, each 5m in diameter, were excavated at a depth of 17m below the ground surface using an Earth Pressure Balance Machine (EPBM).

For the purposes of this paper, the effects of only the first tunnel installed (westbound) was examined (see Fig. 8), and the results are reported only for Murdoch house (Figure 8). The buildings consisted of load-bearing masonry founded upon shallow strip footings. The FEM programme ANSYS (2012) was used and 100 Monte Carlo Simulation loops were executed based on the probabilistic distribution function summarised in Table 2.

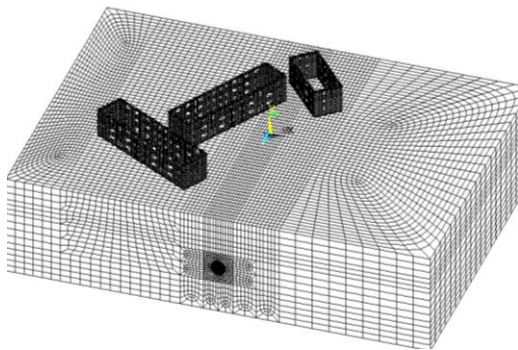


Fig. 8 3D CNM of the case study

For each simulation loop, the state of strain (Eqn. 5) was determined. The 100 Latin Hyper Cube Simulation pairs of β and ε_1 were plotted (Figure 10), as well as the different limiting principal strains. Figure 11 shows a histogram of the subsequent damage categories based on definitions from Table 1. The histogram shows that with a probability of 0.95, the damage at the Murdoch house was categorised as less than "slight". The result can also be illustrated in Figure 12 based on principal strain (Eqn. 5).

Table 2 Uncertainty properties relating to surface buildings (SD=Standard Deviation, N=Normal, U=Uniform)

Parameter	Mean	SD	Lower	Upper	PDF
Masonry					
Young's Modulus (MPa)	3130	1509	1260	5140	N
Density (kg/m ³)	2000	100	1630	2200	N
Poisson's Ratio	0.2	0.04	0.13	0.25	N
Comp. Strength (MPa)	7.4	4.3	1.0	16.0	N
Tensile Strength (MPa)	-	-	0.02 f _c	0.1 f _c	U
Soil-Structure Interface					
Friction Angle	7.4	0.3	4.5	1.1	N
Tensile Strength (MPa)	-	-	0.02 f _c	-0.1 f _c	U
Soil-Structure Interface					
Friction Angle	-	-	0.3	0.6	U

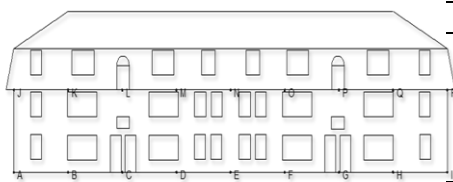


Fig. 9 Murdoch house (the points shows the locations that state of strain were calculated based on them)

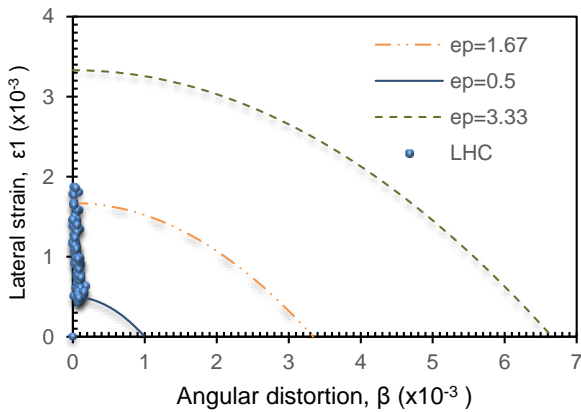


Fig. 10 Plots of 100 LHS pairs of (β, ϵ_1) for the case study

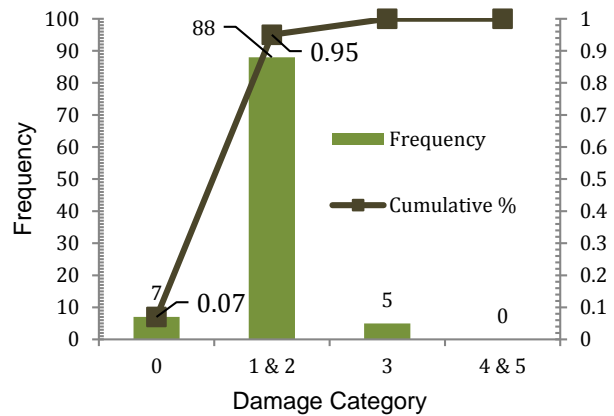


Fig. 11 Histogram of damage categories of case study

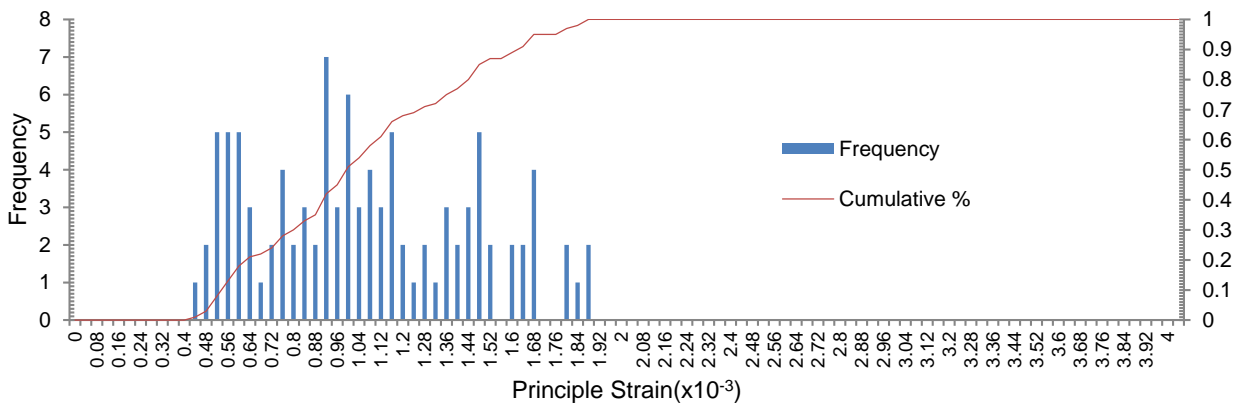


Fig. 12 Reliability curve for assessing the performance of the system

6. Steps remaining to complete the framework

The uncertainty characteristics of the case study are compared to the elements of the framework discussed in this paper in Table 3. The objective of the framework proposed herein was to address all combinations of parameter uncertainty and to assess their effects in the same model, since in a general CNM problem, most of variables summarised in Figure 1 or table 3 are uncertain.

Among the uncertainties summarised in Figure 1, only a limited combinations of uncertainties related to the masonry and soil interaction were partially addressed in the case study. As an example of some of the limits an elastic soil media was considered and the 3D excavated ground model was calibrated with an empirical greenfield formula.

Furthermore, as the parameters of the subsurface including construction variability were all deterministic, the calculation was done for only one specified hazard level. To complete the framework more realistically, it must be examined with a case study that covers all groups of uncertainties. More effort would also be needed to address uncertainties related to the problem's geometry.


7. Conclusions

This paper summarized the initial steps needed to achieve a fully probabilistic framework for the large-scale risk assessment of existing unreinforced masonry buildings subjected to bored tunnelling. All potential variables, which propagate uncertainties to assess vulnerability of masonry building were grouped and discussed. A vulnerability analysis framework based on the concept of performance-based design/assessment (PBD) was proposed to compute the risk. The adapted

approach was examined with a published case study, and a masonry structure relevant to the case was assessed numerically based on proposed probabilistic performance criteria. Since the examined case study does not cover all the potential parameters that contribute in a real CNM, further steps are planned to achieve a robust framework for a general case.

Table 3 The characteristic of the case study discussed in this paper

Parameter/Element		Addressing	Comments				
Potential variables	Subsurface	Problem Dimension		-	3D		
		Epistemic	Modelling		The subsidence trough is calibrated by empirical formula, uncertainty was not addressed in these groups		
			Monitoring				
			Construction variability				
		Aleatory	Soil properties				
			In-situ stress				
			Loading cases				
			Ground water				
			Masonry	Epistemic		Modelling	
	Monitoring						
	Foundation Type			Soil-structure interface concept was considered			
	Aleatory	Material Properties			Openings are modelled deterministically		
		Structural characteristic					
		Loading Cases					
		Foundation types					
		Initial state					
	Geometry	Epistemic	Documentation				
			Foundation arrangement				
			Tunnel route		Rout of tunnel addressed deterministically		
			Wall thickness and arrangement				
Floor thickness							
Opening ratio							
Elements of VA	Damage Classification						
	Fragility curves			One specified hazard level was considered			
	Consequence-damage diagram						
	Performance criteria						


 Partially addressed
 Fully addressed

8. Acknowledgments

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