PREVENTIVE AND CURATIVE MEASURES IN HERITAGE BUILDINGS DUE TO SETTLEMENTS RISK WITH CONSEQUENT DEGRADATION OF THEIR MASONRY WALLS

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ABSTRACT

The foundations movements can be a cause of anomalies in affected buildings. Such situations can have particularly negative implications for heritage buildings that represent a particular cultural and historical value that matters to preserve.

This article intends to present study methodologies for the implementation of preventive and curative measures in existing Heritage Buildings with reinforced concrete structure or mixed structure of reinforced concrete and masonry walls, specifically focusing on the risks of foundation settlements that, in particular, can cause cracking in masonry walls.

Foundation settlements that are considered here relate, especially, to those resulting from the execution of geotechnical works, in the building itself or in its neighbourhood. Methodologies are presented to estimate and limit the damage. Strategies for repair and rehabilitation of walls and reinforcement of foundations of buildings are also presented.

Key-words: Foundation settlements, Masonry walls, Buildings

1. INTRODUCTION

The foundations movements can be a cause of anomalies in affected buildings. Such situations can have particularly negative implications for heritage buildings that represent a particular cultural and historical value that matters to preserve.This article intends to present study methodologies for the implementation of preventive and curative measures in existing Heritage Buildings with reinforced concrete structure or mixed structure of reinforced concrete and masonry walls, specifically focusing on the risks of foundation settlements that, in particular, can cause cracking in masonry walls.

Foundation settlements that are considered here relate, especially, to those resulting from the execution of geotechnical works, in the building itself or in its neighbourhood. Methodologies are

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presented to estimate and limit the damage. The suitability of the limits usually adopted in the design of these works (excavation of tunnels, open excavations, downgrade of the water table, etc.) is discussed, in order to avoid the occurrence of cracking in masonry walls. Strategies for repair and rehabilitation of walls and reinforcement of foundations of buildings are also presented, in view of the occurrence of damage due to the foundation settlements. In this paper existing limiting criteria established to estimate the likely damage to the building are summarized, and a "beam in elastic medium model" that was previously developed is confronted with the above referred existing limiting criteria, and some outcomes of this model, in terms of prevention of damage to the buildings, are here presented.

2. DESCRIPTION OF FOUNDATION SETTLEMENTS AND DEFINITION OF THEIR LIMIT CRITERIA

2.1 Definitions of ground and building movements

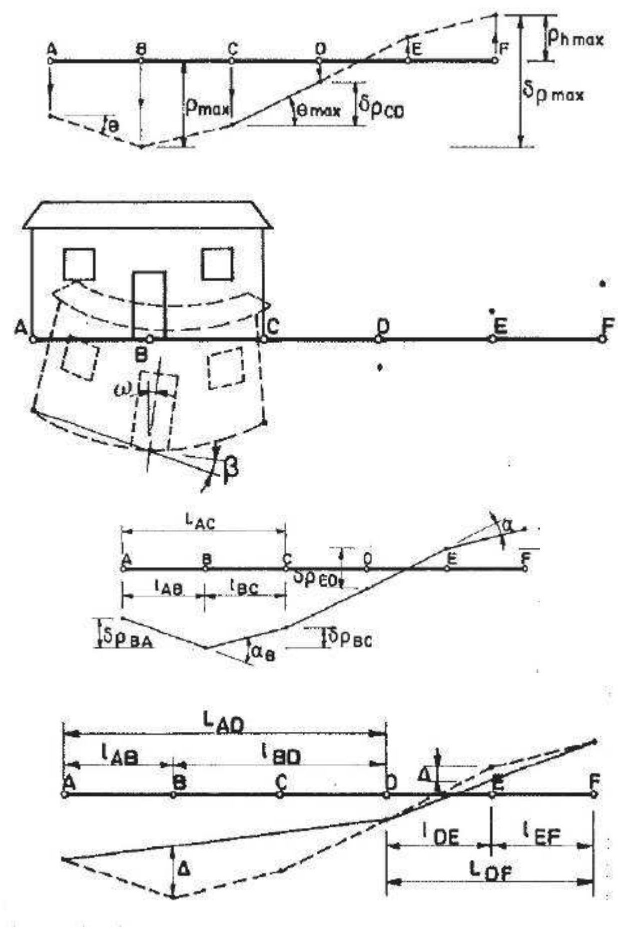
The definitions of ground and building movements that are here given can be applied to majority of situations of damage to the buildings due to ground movements, regardless of the cause of movement, and can, in particular, be applied to ground movements resulting from tunnelling and deep excavations and the way they affect overlying structures and services.

Skempton and MacDonald (1956) examined records of nearly 100 buildings (mainly infilled steel or reinforced concrete framed, but a few load bearing walls [1]). The damage was correlated with angular distortion (δ/L). But there were some difficulties to relate damage with angular distortion as a unique control parameter of the foundation movement to be considered, and angular distortion δ/L implicitly assumed the building deforming in shear and without distinguishing between hogging and sagging. There was a clear evidence that buildings do not only deform in shear as Burland and Wroth (1974) explained it. A more fundamental approach was required in assessing limiting deformations, and it was first necessary to set out definitions of foundation movement, which do not make assumptions about the mode of deformation of the superstructure. In the following, it is presented the definitions of ground and foundation movement, according to Burland and Wroth (1974 [1]), which the following are highlighted due to their interest for the subjects treated herein (see Figure 1): relative deflection, Δ, and deflection coefficient, Δ/L; tilt, ω; and relative rotation, β (angular distortion).

2.2 General aspects about damage of buildings due to foundations movements

Foundation movements, particularly related to the tunnelling and deep excavations, can significantly affect overlying building structure. The experience of observation reveals that relatively low differential settlements can produce slightly cracking in the masonry facade walls.

Masonry is a composite material, which consists of elastic brittle blocks or bricks linked through mortar joints. Due to their low bond strength, particular in tension, these joints act as planes of weakness. Depending on the degree of compression present, failure can occur in the joints alone or as a combined block/brick-joint failure, so the joint strength and orientation has in important influence on the cracking of masonry [13]. Experimental research studies had shown that the shear resistance can be assumed to reduce approximately to zero between the situation of high compressive stress and tensile stress; and that the joint elements have relatively high compression capacity and low tensile resistance (flexure tensile stress), and a shear capacity which is a function of the imposed compression and the bond strength. In fact, the complex triaxial stress state produced by mortar-block and mortar-concrete element (beams and slabs) interaction, and the influence of bonding pattern of the wall, makes difficult the understanding of the effective behaviour of the block/brick and the mortar in the masonry, and this last with the concrete beam/slab. The non-linear characteristics of this type of masonry (block/brick) result most probably from the local failure and slip that occur in the joints and the non-linear deformation typical of the joints under shear and compression. So, these characteristics are recommended to be taken in account when estimating the mechanical parameters, particularly for numerical simulations of wall-beam/slab behaviour; but due to their complexity in defining models for structure-soil interaction, some simplified assumptions about these characteristics can be adopted.



Settlement is denoted by the symbol ρ (or S) and implies that the displacement is downwards. If the displacement is upwards, it is termed heave and denoted by ρ_h (or S_h)
Differential or relative settlement (or heave) is denoted by δρ (or δS). In Figure l the settlement of C relative to D is denoted $\delta\rho_{CD}$ and is taken as positive. The settlement of D relative to C is denoted by $\delta\rho_{DC}$; which equals $-\delta\rho_{CD}$. Maximum differential settlement is denoted by $\delta\rho_{max}$.

Rotation is denoted by θ (Figure 1) and is used to describe the change in gradient of the straight joining two reference points embedded in the foundation or ground.
Tilt is denoted by ω and normally describes the rigid body rotation of the whole superstructure or of a well-defined part of it. Normally it is not possible to ascertain the tilt unless details of the superstructure and behaviour are known. Even then it can be difficult when the structure itself flexes. Figure shows diagrammatically the tilt building overlying points ABC.

Relative rotation is denoted by β and describes the rotation of the straight line joining two reference points relative to the tilt (Figure 1), Note that the definition of the angular distortion (see Figure 1) is identical to the relative rotation β.
Angular strain is denoted by α.

Deflection ratio or coefficient (DR) - sagging ratio (DRS) or hogging ratio (DRh) - is determined as Δ/L, where Δ is the relative deflection (relative sag or hog) and L is the length of structure subjected to that deflection.
Relative deflection, Δ, is the maximum displacement relative to the straight line connecting two reference points a distance L apart.
Relative sag corresponds to upward concavity (as at B in Figure 1 - Δ is positive).
Relative hog corresponds to downward concavity (as at E in Figure 1 - Δ is negative).

Figure 1. – Definitions of ground and foundation movement, according to Burland and Wroth (1974 [1])

In foundation movements (see definitions in Figure 1), the interaction of the concrete structure buildings with the ground and their consequent possible damage are in fact very complex to analyse in terms of the deformations of foundation soil and the building structure with their infill masonry walls, as well as the stress in the different parts of the buildings, which can be sufficiently high and lead to cracking to the building elements. In order to deal with such complex situation, generally, it is assumed, for simplification of the problem, that the referred damage, due mainly to cracking of masonry, can be associated to the tensile strains that are induced in the masonry walls of those buildings (Burland and Wroth, 1974 [1]), with the building modelled as elastic deep beams, as it will be explained in detail in the following. Research studies have revealed that masonry walls could follow, reasonably well, the concave and convex deformations imposed on their support (foundation beam), ever since masonry joints (connecting the blocks) can have adequate resistant capacity [12]; it was found that, for large imposed curvature to the wall with openings, the wall is more susceptible to cracking if there is an imposition of a concave deformation (hogging), which can generate cracking in vertical masonry joints, than a convex one [12].

2.3 Criteria based on concept of critical tensile strain

2.3.1 Model based on the concept of critical tensile strain (Burland and Wroth, 1974 [1])

Burland and Wroth observed that buildings generally become unserviceable before they present signs of a risk of structural collapse. Most damage to walls, cladding and finishes, manifests as cracking which results from tensile strains. Burland and Wroth noted that the locally determined tensile strains, at which cracking became visible, was reasonably well defined and independent both of the tensile strength of the masonry and blockwork and of the form of loading of the wall; whether it was subjected to racking in shear or in-plane bending. They concluded that the value of critical tensile strain, ε_{crit}, varied between about 0.05% and 0.1% and suggested the use of an average value of 0.075%. So they concluded for the need of changing the established empirical deflection criteria and analysing the essential causes of damage to walls and their finishes in terms of cracking. Their analysis was based on the work carried out at the Building Research Establishment [1]: BRE large scale tests on composite action between masonry walls and their supporting beams - Burhouse, 1969; and BRE tests on the stiffness and strength of masonry infilled frames (Mainstone, 1971). They stressed that ε_{crit} is significantly larger than the strain at which tensile failure occurs. It is also an average strain measured over a gauge length of about a metre.

Burland and Wroth then applied the concept of critical tensile strain, ε_{crit}, to evaluate the limiting displacements of simple weightless elastic beams of length L and height H. Even though real buildings are much more complex, this study analysed a number of important features that control limiting values of Δ/L.

2.3.2 Relevant models based on the progression of the concept of tensile strain to limit extension (Burland (1977); Boscardin e Cording (1989))

Later, it was noted (Burland *et al* (1977 [2])) that the critical tensile strain causing the onset of visible cracking is not a fundamental material property. The onset of visible cracking represents a level of damage of about Category 1 (see definition in cap 3).
It would be better to think of the tensile strain as a serviceability parameter, the magnitude of which can be chosen to take account of different materials and serviceability limit states. Hence, they replaced ε_{crit} by ε_{lim} (limiting tensile strain). It was considered, also, the likely progression of damage after the initiating of visible cracking. If realistic estimates are to be made of allowable relative deflections of buildings, it is necessary to take some account of their stiffness.
The ground movements resulting from tunnelling and from excavations often include significant horizontal components of displacement. These have to be taken in account when assessing impacts on buildings and services. Boscardin and Cording introduced two important advances in this context. Both in the Burland and Wroth (1974 [1]) and Boscardin and Cording (1989 [3]) approaches, it was intended to relate crack damage with the level of maximum principal tensile strain developed in the building structure. A relationship between category of damage and limiting tensile strain (ε_{lim}) was presented (Boscardin and Cording, 1989 [3]) for elastic building models.

2.3.3 Other Approaches *(Model of beam in elastic medium; Miranda Dias, 1991)*

The analogy proposed in this simplified model start from the idea that the superstructure based on the ground can be closely assimilated to a beam in elastic medium (Miranda Dias, 1991 [11]). This analogy approximately can describe the real behaviour of the soil-superstructure ensemble, and it may be of some use for the general understanding of the behaviour of the buildings subjected to movements of the foundations. It is accepted, as in the model presented before (Burland, 1974; in 2.3.1), that the visible crack in masonry walls is associated with the critical tensile strain, ε_{crit}. It is intended to limit, in the beam model, the tensile strain in the extreme lower fibre of its section, in half span, when subjected to a vertical load concentrated in half span. In this case the model is particularly suitable for representing the behaviour of buildings, in which global deformed the respective neutral line occupies, roughly, a central position in the section. This may happen in buildings based on brick masonry walls or other fragile material. The theory of the beams in the elastic medium defines the parameter elastic length λL (being L the length of the beam) which allows to account for the relationship between the rigidity of the beam and the foundation soil [9].

The λ value is given by: $\lambda = \left[\frac{k}{4\mathrm{EI}}\right]^{\frac{1}{4}}$ And the elastic length λL is: $\lambda\mathrm{L} = \left[\frac{k \cdot L^4}{4\mathrm{EI}}\right]^{\frac{1}{4}}$

k - Winkler reaction module which depends, among other factors, of the values of E_s (modulus of deformability) and μ_s (soil Poisson coefficient), for b-beam width equal 1(Unitarian value);

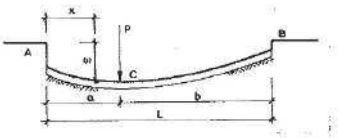
E - Modulus of elasticity;
I - Moment of inertia of the beam
k can be calculated with the following expression [10]:
E_s - Modulus of elasticity of the soil;
k_0 - Soil reaction module
μ_s - Soil Poisson coefficient

$$\mathrm{k} = k_0.\mathrm{b} = 70.\left[\frac{E_s.b^4}{\mathrm{EI}}\right]^{\frac{1}{12}} \cdot \left(\frac{E_s}{1-{\mu_s}^2}\right)$$

The variables involved in definition of the λL parameter (elastic length λL) suggest the establishment of a parallel between its significance in the theory of the beam in the elastic medium and the model soil-structure that is intended to be analysed. In fact, in this model, the term *EI* can translate in an approximate way the characteristics of the building, as well as the variable k can translate characteristics of rigidity of the foundation soil on which the building rests. The values assumed by λL may, thus, classify in a synthetic way the different conditions of deformability of the beam in the elastic medium, so three large classification groups were determined:

$\lambda\mathrm{L} < \pi/4$ → It is classified as a short beam (of high rigidity); being more important the deformation of the foundation than that of the beam;
$\pi/4 < \lambda\mathrm{L} < \pi$ → It is classified as an intermediate beam;
$\lambda\mathrm{L} > \pi$ → It is classified as a long beam, which can be assimilated to an infinite beam for calculation purposes.

In the following a simplified estimation of Δ/L is deducted, considering the analogy proposed in the simplified model described above.



L- beam length
λl - elastic length
w – deflection at the point of abscissa x
P – concentrated load

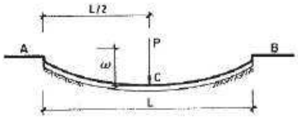
Deflection w (at mid-span [9])

$$\mathrm{w} = \frac{P.\lambda}{k} \cdot \frac{1}{sinh^2\ \lambda L.\sin^2\lambda L} \cdot \left\{\left(2\cdot\cosh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\right)\cdot\left(\sinh\lambda L\cdot\cos\frac{\lambda L}{2}\cdot\cosh\frac{\lambda L}{2} - \sin\lambda L\cdot\cosh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\right) + \left(\cosh\frac{\lambda L}{2}\cdot\sin\frac{\lambda L}{2}\cdot\sin\frac{\lambda L}{2}\cdot\sinh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\right)\cdot\left[\sinh\lambda L\cdot\left(\sin\frac{\lambda L}{2}\cdot\cosh\frac{\lambda L}{2} - \cos\frac{\lambda L}{2}\cdot\sinh\frac{\lambda L}{2}\right) + \sin\lambda L\cdot\left(\sinh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\cdot\cosh\frac{\lambda L}{2}\cdot\sin\frac{\lambda L}{2}\right)\right]\right\}$$

Moment M (at mid-span [9])

$$\mathrm{M} = \frac{P}{2.\lambda} \cdot \frac{1}{sinh^2\ \lambda L\cdot\sin^2\lambda L} \cdot \left\{\left(2\cdot\sinh\frac{\lambda L}{2}\cdot\sin\frac{\lambda L}{2}\right)\cdot\left(\sinh\lambda L\cdot\cos\frac{\lambda L}{2}\cdot\cosh\frac{\lambda L}{2} - \sin\lambda L\cdot\cosh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\right) + \left(\cosh\frac{\lambda L}{2}\cdot\sin\frac{\lambda L}{2} - \sinh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\right)\cdot\left[\sinh\lambda L\cdot\left(\sin\frac{\lambda L}{2}\cdot\cosh\frac{\lambda L}{2} - \cos\frac{\lambda L}{2}\cdot\sinh\frac{\lambda L}{2}\right) + \sin\lambda L\cdot\left(\sinh\frac{\lambda L}{2}\cdot\cos\frac{\lambda L}{2}\cdot - \cosh\frac{\lambda L}{2}\cdot\sin\frac{\lambda L}{2}\right)\right]\right\}$$

Concentrated load at mid span



$\mathrm{k} = k_0.\mathrm{b}$
k_0 – soil reaction module
b – beam width

$$\lambda = \left[\frac{k}{4\mathrm{EI}}\right]^{\frac{1}{4}}$$

Deflection (1/2 span)

$w = \frac{\mathrm{P.\lambda}}{\mathrm{k}}\ .\alpha_w$ (a=b=L/2)

if : $w' = \mathrm{w.k.L} = \frac{\mathrm{P.\lambda}}{\mathrm{k}}\ .\alpha_w.\mathrm{k}$

$$w' = \mathrm{P.(\lambda.L)}.\alpha_w$$

Flexural bending moment (1/2 span) $M = \frac{\mathrm{P}}{2\lambda}\ .\alpha_M$ (a=b=L/2)

$M' = \frac{M}{L} = \frac{\mathrm{P}}{2.\lambda}\ .\alpha_M.\frac{1}{L}$

$$M' = \frac{P}{2.(\lambda.L)}\ .\alpha_M$$

Assuming the neutral axis in the middle of the section, follows:

$$\epsilon = \frac{\sigma}{E} = \frac{MH}{2.\,\mathrm{EI}}$$

ϵ - tension strain in the lower extreme fibre
H - beam height

Supposing that $\Delta \approx w$

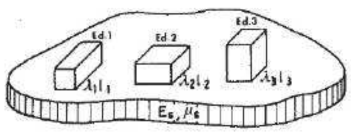
$$\frac{\Delta}{L\epsilon} = \frac{w}{M}\cdot\frac{2\mathrm{EI}}{\mathrm{LH}} = \frac{w'}{kL}\cdot\frac{1}{M'\cdot L}\cdot\frac{2\mathrm{EI}}{\mathrm{LH}} = \frac{w'}{M'}\cdot\frac{1}{kL^2}\cdot\frac{2\mathrm{EI}}{\mathrm{LH}} = \frac{w'}{2M'}\cdot\frac{4EI}{HkL^3} = \frac{w'}{2M'}\cdot\frac{4EI}{kL^4}\cdot\frac{L}{H}$$

$$\frac{\Delta}{L\epsilon} = \frac{w'}{2M'}\cdot\frac{1}{(\lambda L)^4}\cdot\frac{L}{H}$$

If $\epsilon = \epsilon_{crit}$ $\quad \frac{\Delta}{L\epsilon_{crit}} = \frac{w'}{2M'}\cdot\frac{1}{(\lambda L)^4}\cdot\frac{L}{H}$

If $\epsilon_{crit} = 0.075 \times 10^{-2}$ (ϵ_{crit} - critical tension strain in the lower extreme fibre: 0.075%)

$$\frac{\Delta}{L} = \frac{3w'}{4M'}\cdot\frac{1}{(\lambda L)^4}\cdot\frac{L}{H}\cdot 10^{-3}$$



Ed1 – Analogy with the model of long beam

Ed2 – Analogy with the model of intermediate beam

Ed3 – Analogy with the model of short beam

Figure 2. – Behaviour analogy between buildings with different dimensions and the models of beams (with different values of elastic length λL) in elastic medium (with different values of E_s (soil elastic modulus) and μ_s (soil Poisson coefficient)) and consequently of the Winkler reaction module, k, whose estimation, particularly, depends on these two values (E_s and μ_s))

Taking in account that the different construction solutions for the buildings (buildings of reinforced concrete structure, buildings of resistant masonry, buildings with mixed structure of reinforced concrete and resistant masonry, etc.) present different stiffness characteristics, it can be admitted that these solutions present different values of elastic length λL.

The analysis based in the model of beam in elastic medium (Miranda Dias, 1991, [11]), allows to suppose that the limiting value for the deflection coefficient Δ/L grows with the decrease of the value of the elastic length λL (assuming $\epsilon_{crit} = 0.075 \times 10^{-2}$ (0.075%)); this criterion is less severe for situations where the deformability characteristics of the foundation soil has more importance than the deformability characteristics of the building (which is the case of "short beam model": $\lambda L < \pi/4$) analysed here. In that analysis, for $\lambda L = \pi$ (near infinite beam), the values of Δ/L are near the values of the criterion of Burland and Wroth (1974). And the limiting values for the coefficient of deflection Δ/L in buildings with masonry walls penalize more the buildings founded on soils such as sand and hard clay than in others like the plastic clay (due to the values of E_s). It should be noted that the values of the criterion of Burland and Wroth (1974) are conditioned by the ratio L/H, being L and H respectively length and height of the wall, while in the model of beam in elastic medium [11] such dimensions represent the length and the height of a building under the conditions expressed here.

3. CLASSIFICATION OF DAMAGE DUE TO SETTLEMENTS OF FOUNDATIONS

As a consequence of settlements, many buildings experienced damage, and it was important to establish a system of classifying damage. The classification proposed by Burland (1995 [4]) is based on ease of repair, and is developed from a large number of other studies; it applies only to masonry and blockwork; it relates to visible damage at a given time and not its cause or possible progression – these have to be considered separately; classification is not based on crack width alone – it is the ease of repair which is the key factor; more stringent criteria may be necessary where cracking could lead to corrosion, penetration of harmful liquids or gasses or structural failure (in the case of a heritage building, generally, that criteria should be particularly more stringent). Categories 0, 1 and 2 represent aesthetic damage; categories 3 and 4 serviceability damage; and categorie 5 stability damage. A three-stage approach for classification of damage was proposed by Burland (1995 [4]).

4. RESPONSE OF BUILDINGS TO EXECUTION OF GEOTECHNICAL WORKS AND RESPECTIVE LIMITS IN THE DESIGN

4.1 **Response of buildings to execution of geotechnical works, in the building itself or in its neighbourhood** (ex: tunnelling)

4.1.1 General

Foundation settlements that are considered here relate, especially, to those resulting from the execution of geotechnical works, in the building itself or in its neighbourhood. Methodologies are following presented to estimate and limit the damage.

Current design procedures hardly model detailed aspects of the mechanisms that cause settlement damage in buildings with masonry walls. To assess settlement-induced damage to buildings, some approaches are based on a finite element method, in which the building, the ground and the tunnelling processes are combined in a numerical model. These models are based on the use of two or three-dimensional analysis to model tunnel installation and consequent settlement-induced damage to an overlying building. These analyses confirm that soil-structure interaction effects have an important impact on the predicted damage to these buildings.

4.1.2 Building structure deformation according to the vertical and horizontal greenfield ground movements (Mair & Taylor, 1997)

Current assessment methods are generally based on a two-stage process. Firstly, the ground settlements, at an equivalent site where there are no buildings, are estimated ("greenfield site" settlements). These displacements are then imposed on a model of the building structure for an assessment of the expected damage. The estimation of the risk of damage to buildings typically involves assuming that the structure deforms according to the vertical and horizontal greenfield ground movements. The greenfield ground movements were analysed, through to extensive field measurements and centrifuge studies (Mair, 1996 [5]); the presence of a structure alters these movements (soil-structure interaction); the modification to these ground movements can result in smaller distortions and levels of damage to the buildings than those predicted before.

4.1.3 ***Parametric finite element analysis*** (Potts & Addenbrooke (1997))

Potts & Addenbrooke (1997 [6]) conducted a parametric finite element analysis to investigate the response of buildings to tunnelling and introduced design charts to consider the influence of the building's own stiffness, thus leading, to new improved predictions of tunnel-induced deformation. Their relative stiffness approach was based on a parametric study using plane-strain finite element (FE) analyses in which the building was modelled by weightless elastic beams. It was present the results of both two- and three-dimensional parametric FE studies that extended their building model to include a wider variety of building features such as building weight, the nature of the soil–structure interface, and the building dimension in the direction of the tunnel axis. By incorporating these additional building features into their approach, it is shown how the relative stiffness expressions can be modified to be dimensionless.

5. STRATEGIES FOR PREVENTION, REPAIR AND REHABILITATION OF WALLS AND REINFORCEMENT OF FOUNDATIONS

5.1.1 General

Strategies for repair and rehabilitation of walls and reinforcement of foundations of buildings are also presented, in view of the occurrence of damage due to the foundation settlements. An assessment of potential damage is particularly important when the buildings are of masonry construction, in which case relatively small differential settlements can lead to the development of unsightly cracking in the walls and facades. If the extent of the predicted damage is unacceptable, particularly in the case of a heritage building, then appropriate action needs to be taken, for example modifications to the design or the specification of settlement control procedures such as compensation grouting.

5.1.2 Prevention related to foundations

According to Burland (1995), the proposed protective measures are presented in the following. Before considering surface measures, tunnelling procedures should be examined. These tackle the root cause of the problem and may prove much less costly and disruptive than near surface measures. In other approach (CIB 2015 [11]) for prevention related to the foundations, in order to avoid the damage of masonry walls, the following measures were presented:

- Evaluating soil parameters: increase the geotechnical investigations;
- Observe dimensions and shapes of the buildings: provide joints;
- Effect of: long dimensions in plan view, sudden change of shape, very different loads, changes in soil and foundation types, different construction periods of adjacent buildings;
- Attention to existence of soft/deep soil layers, fluctuations of the water table and leaking of the drainage system that saturates the soil around shallow foundations.

5.1.3 Prevention related to masonry walls

According to EN 1996-1-1:2005 (EC6), masonry structure shall be designed and constructed so as not to exceed the Serviceability Limit State (EN 1996-1-1:2005 - Section 7, Serviceability Limit State, 7.1 General). Deflections that might adversely affect partitions, finishing (including added materials) or technical equipment, or might impair water-tightness should be checked. The serviceability of masonry members should not be impaired by the behaviour of other structural elements, such as deformations of floors or walls.

Based on the values of the mechanical characteristics, recommended in the EN 1996-1-1 :2005 (EC6, [15]), for different combinations between the various types of blocks and mortar joints, estimated reference values are following presented in Table 1 (for situations of buildings in a usual foundation soil; not applicable to situations of tunnelling); in particular, a range of values with upper and lower extremes for the relationship (f_w/L) between the cracking deflection f_w, occurring after the construction of the walls, and the span L, based in the model of beam in elastic medium presented in 2.3.3, [11].

The reference values for f_w/L were determined through a simplified model which assumes that the building wall cracking deflection was conditioned by the limiting values of deflection coefficient (Δ/L) and, therefore, the referred values of f_w/L were approximated to the estimated limiting values of Δ/L, calculated for two values of the elastic length λL, $\pi/3$ and $\pi/4$; considering the definition of λL ($\lambda L=(k.L^4 / EI)^{1/4}$), for a fixed type of soil and length contact (L) of the building resting in soil (that means a fixed value of $k.L^4$), that corresponds to a rigidity value (E_1I_1) of the building with elastic length λL of $\pi/4$ that is near 3 times superior the rigidity value (E_2I_2) of the building with elastic length λL of $\pi/3$; and the critical tension strain in the lower extreme fibre (ϵ_{crit}) is estimated in relation with the characteristic initial shear strength of the masonry (f_{vko}) and the Modulus of Elasticity in tension (E_t), and is presented in Table 1 for the example of mortar class M1-M2 ($\epsilon_{crit} = (f_{vk0}/2)/E_t$)). It should be noted that these values of f_w/L should only be regarded as guiding values.

The global analysis of the results in Table 1 allows to suppose that the reference values for the relationship (f_w/L) lead to less severe limitations as values of the elastic length λL decreases (from $\pi/3$ to $\pi/4$) and as L/H or mortar joints class resistance increases. Such severity increases with the decrease of the proportion of holes in the blocks. Indeed, the evaluation of the deformation of the foundation elements that lead to cracking of masonry walls need to be soundly based, for each specific case, on the consideration of the deformation characteristics of the building and the soil and its interaction, as well as taking in due account the results of the observation experience of buildings subjected to settlement of the foundations.

5.1.4 Repair and rehabilitation of walls and reinforcement of foundations

Burland (1995) presented a range of surface or near surface measures including strengthening the ground, structural jacking, underpinning and strengthening the building. In other approach (CIB 2015 [11]), for repair of the foundations, in order to avoid the damage of masonry walls, the following measures were presented:

- Consolidation of soils and/or increase stiffness of foundation elements;
- Insertion of joints, allowing the building parts to perform as independent rigid bodies;
- Use of deep foundation when the water table fluctuates or when there are soft/deep soil layers;
- Fix the drainage system.

For repair of damage of masonry walls related to foundation differential settlements, some methods and their applicability are following presented (CIB 2015 [16]):

1-Raking and Re-pointing

- Usually applied to cracks localized in the mortar joints, especially in heritage buildings.

2- Re-construction of selected areas

- Usually applied to restore structural integrity, including demolition and re-building of the damaged area;

3 -Resin injection

- Usually applied to cracks in masonry units and to mortar joints.

Table 1. Reference values for the relationship between cracking deflection of the wall and span for two typical values of span/height of masonry walls

Type of masonry unit	**Reference values for the relationship f_w/L** (Cracking deflection of the wall / span of the wall)										
			Mortar Joint Classes (EC6 - EN 1996-1-1:2005)								
	Group of units (3)	**K**	**M1 to M2**			**M2,5 to M9**			**M10 to M20**		
		Constant for masonry **f_k**, (taken from EC6 -Table 3.3) **ϵ_{crit} (x 10^{-5})** mortar class M1-M2	Masonry Elasticity modulus **Ec** Initial shear strength **f_{vko}** (MPa)	Cracking deflection /span **f_w/L** For values of Elastic Length λL **λL=π/4** and **λL= π/3**		Masonry Elasticity modulus **Ec** Initial shear strength **f_{vko}** (MPa)	Cracking deflection /span **f_w/L** For values of Elastic Length λL **λL=π/4** and **λL= π/3**		Masonry Elasticity modulus **Ec** Initial shear strength **f_{vko}** (MPa)	Cracking deflection /span **f_w/L** For values of Elastic Length λL **λL=π/4** and **λL= π/3**	
				L/H=1.5	**L/H=3.5**		**L/H=1.5**	**L/H=3.5**		**L/H=1.5**	**L/H=3.5**
Clay	**Group 1**	0.55 4.8	4265 0.10	1/1407 1/9119	1/603 1/3909	6285 0.20	1/1333 1/8639	1/571 1/3703	13273 0.30	1/1194 1/7740	1/512 1/3317
	Group 2	0.45 5.9	3318 0.10	1/1151 1/7461	1/494 1/3198	6285 0.20	1/1090 1/7068	1/468 1/3030	10860 0.30	1/977 1/6333	1/419 1/2714
	Group 3	0.35 7.5	3318 0.10	1/895 1/5803	1/384 1/2487	9877 0.20	1/848 1/5498	1/364 1/2356	8447 0.30	1/760 1/4926	1/326 1/2111
	Group 4	0.35 7.5	5213 0.10	1/895 1/5803	1/384 1/2487	8081 0.20	1/848 1/5498	1/364 1/2356	8447 0.30	1/760 1/4926	1/326 1/2111
Calcium silicate	**Group 1**	0.55 4.8	4265 0.10	1/1407 1/9119	1/603 1/3909	9877 0.15	1/1777 11519	1/762 1/4937	13273 0.20	1/1791 1/11610	1/768 1/4976
	Group 2	0.45 5.9	5213 -	1/1151 1/7461	1/494 1/3198	5253 0.15	1/1454 1/9424	1/623 1/4039	10860 0.20	1/1465 1/9499	1/628 1/4071
Aggregate Concrete	**Group 1**	0.55 -	2773 -	-	-	4669 -	-	-	13273 0.20	1/1791 1/11610	1/768 1/4976
	Group 2	0.45 -	2465 -	-	-	4086 -	-	-	7059 0.20	1/953 1/6174	1/409 1/2646
	Group 3	0.40 -	2157 -	-	-	9877 -	-	-	6275 0.20	1/847 1/5488	1/363 1/2352
	Group 4	0.35 -	5213 -	-	-	8081 -	-	-	5490 0.20	1/741 1/4802	1/318 1/2058
Autoclaved Aerated	**Group 1**	0.55 -	4265 0.10	-	-	8081 0.15	1/1777 1/11519	1/762 1/4937	13273 -	-	-
Manufactured stone	**Group 1**	0.45 5.9	4265 0.10	1/1151 1/7461	1/494 3198	6285 -	-	-	10860 -	-	-
Dimensioned natural stone	**Group 1**	0.45 5.9	3318 0.10	1/1151 1/7461	1/494 1/3198	6285 0.20	1/1090 1/7068	1/468 1/3030	10860 0.30	1/977 1/6333	1/419 1/2714

(1)- f_{vko} – characteristic initial shear strength, under zero compressive stress (initial cohesion), that is the resistance of block-mortar joint for a null compression based on EN 1052-3 or 1052-4 or on the basis of the Table 3.4 of EC6 (EN 1996-1-1:2005); values here of f_{vko} are taken from Table 3.4 of EC6;

(2)- masonry group classification according to EC6, Table 3.1: Group 1-massive blocks or with percentage of vertical drilling less than 25% and subject to certain conditions; Group 2-blocks or bricks with percentage of vertical drilling more than 25% but less than 45% or 60% (blocks of concrete); Group 3-blocks or bricks with vertical drilling percentage greater than 25% but less than 70%; Group 4-blocks with horizontal drilling percentage less than 70% and subject to certain conditions

(3)- two values representing the ratio between the length L and height H of the walls (L/H = 1.5 and L/H = 3.5) were here chosen; it should be noted that these same values of L/H served as reference values in ISO 4356 [14] for the assessment of the behaviour of walls face to the imposed deformations;

(4)-the value of the Modulus of Elasticity in compression E_c ($E_c = K_E.f_k$; K_E =1000, see EC6, 3.7.2) of the masonry, adopting on their determination, by simplification, a value of f_b equal to f_{vk} /0.065 and f_k= K.$f_b^{0.7}$. $f_m^{0.3}$, for the case of use of a masonry mortar joint of a class M1 to M2, M2.5 to M9 and M10 to M20, with adopted values, respectively, of 2 MPa, 10 MPa and 20 MPa; f_b is the normalised mean compressive strength of the units (adopted a value of $f_b = f_{vk}$ /0.065; the adopted simplified values of limit shear resistance f_{vk} are 1.2, 1.5 and 1.7, respectively, in the case of use of a masonry mortar joint of a class M1 to M2, M2.5 to M9 and M10 to M20; K is a constant value taken from EC6 (Table 3.3) – and here, in Table 1, are included the values for use with general purpose;

(5) here it is assumed that $\epsilon_{crit} = (f_{vk0}/2)/E_t$, admitting a Modulus of Elasticity in tension E_t of $E_c/3$ reduced of 60%, taking into account that it is the calculation of long-term deformations (EC6 -3.7.2).

6. CONCLUSIONS

The present paper described here definitions of foundation movement and the concept of limiting tensile strain, as well as the identification of crucial aspects of behaviour of buildings affected by settlements of foundation and the categorisation of their damage. The importance of building stiffness in modifying deformations was here highlighted, strategies for prevention, repair, and rehabilitation of walls and reinforcement of foundations was presented. It is important to make, as possible, a realistic approach to assess the risk of damage of heritage buildings affected by settlements of foundation as well to study monitored case studies of heritage building response.

ACKNOLEGDMENTS

LNEC Planned Research Programme (P2I) for the period 2013-2020 (P2I Project "COREAP" - Service life, conservation and rehabilitation of walls of buildings with relevant patrimonial value") has funded the present study. The collaboration in this article of LNEC Senior Researcher João Barradas is gratefully acknowledged.

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