



WALLS OF MASONRY BUILDINGS SUBJECTED TO IRREGULAR SETTLEMENTS – PROPOSITION OF A SUPPLEMENT TO EUROCODE 6

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Abstract

Analysis of load-bearing walls of masonry buildings subjected to influences connected with settlements, especially irregular ground movements constitutes much more complex problem. Final Draft of Eurocode 7 (2004) gives only general requirements and recommendations concerning design methods of such loaded structures. Also Eurocode 6 (2003) contains no information or procedures for design of masonry structures subjected to vertical ground movements.

Therefore, on the basis of Polish tradition and large experience and on behalf of Polish Standard Institution the method of limit state analysis, based on in-plane state of deformation of the wall for masonry buildings subjected to irregular settlements has been presented in this paper and proposed as a supplement to Eurocode 6 (2003).

Key Words

Masonry structures, stiffening walls, irregular settlement, Serviceability Limit States.

1 Introduction

Design of buildings and structures always starts from foundations calculation. According to existing regulations and recommendations foundations of buildings should be calculated taking into consideration many different factors giving loads or having an influence on structure. Defining the design situations and limit states, according to Eurocode 7 (2004), the following factors should be considered:

- site conditions with respect to overall stability and ground movements;
- nature and size of the building and its parts,
- conditions with regard to its surroundings (e.g.: neighbouring existing or new erected buildings and structures, new services, planned changes of local environment, hazardous chemicals or effects of chemical corrosion, weathering, freezing;

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- ground conditions, especially ground stratification and the effect of local topography;
- ground-water conditions, especially changing the ground water level, neighbouring of trees, effects of floods or long duration droughts;
- regional seismicity or other types of dynamic influences e.g. connected with human activity like mine workings, caves or other underground structures, coal or copper mining, urban transport;
- influence of the environment (hydrology, surface water, subsidence, seasonal changes of temperature and moisture).

The correct taking into consideration all factors described above in design practice is usually not so simple. Many of them have typically random character. It is rather not possible to determine in details the loads or displacements in many cases in real design situation. Design engineers who require information can take it from available different publications and works, e.g. like BRE reports (1984, 1989a, 1989b or 1991).

In case of masonry buildings and structures the situation is much more complicated. Many buildings have problems with cracks produced by irregular ground movements – see Greenspan et al (1980), Freeman et al (1994), Parkinson et al (1996), or Cook et al (1995). Except for problems with correct determination of geotechnical conditions and influences, there are no clear regulations and recommendations concerning design methods of load-bearing structures – masonry stiffening (shear) walls. Even when the internal forces or stresses are correctly calculated, in masonry standards – including last draft of Eurocode 6 (2003) – they are not methods for checking the ultimate and serviceability limit states.

Problem of buildings walls analysis, especially built of masonry and situated on terrains subjected to differential settlements is one of main research and theoretical tasks in Poland for many years. Therefore we have a considerable number of test data of vertically sheared masonry walls and data obtained from monitoring of hundreds existing buildings. Based of these results and information a method of limit state analysis through checking of the serviceability limit state for masonry load-bearing shear walls was elaborated. This method was already widely presented by author – see Kubica (1998a, 1998b, 2002 and 2003) and introduced into Polish Masonry Code PN-B-03002:1999 (1999). This new regulations can be also useful in design practice in other European countries. Therefore in behalf of Polish Standards Institution it is proposed as a supplement to Eurocode 6 (2003).

2 Masonry buildings subjected to differential settlements

2.1 General

In analysis of masonry stiffening (shear) walls both short-term and long-term design situations needed to be considered. Loads in form of irregular settlements should be treated as components of basic load combination. Only vertical deformations connected with appearance of accidental situation like flood, may be treated as an accidental loads. Designer must take the suitable decision. In case of settlements, the load state of analysed masonry wall is connected with vertical ground movements appearance in characteristic points of buildings foundation – as it is shown in Fig. 1. Method of determination of these vertical displacements (u_i) should be taken from geotechnical analysis. Of course, if it is possible, displacements can be determined taking into consideration the building-subsoil interaction. In masonry buildings with wall structural configuration these characteristic points are in places of connections of analysed wall with transverse load-bearing walls (in Fig.1 – points marked as A_1 to A_n). After determination of ground deformation parameters the problem of limit states verification and checking if established state of strains is hazardous for safe

exploitation of analysed building or structure still remains. Practically, there is low number of publications and other detailed requirements giving admissible accepted values of differential settlements – e.g. see Polshin et al (1987).

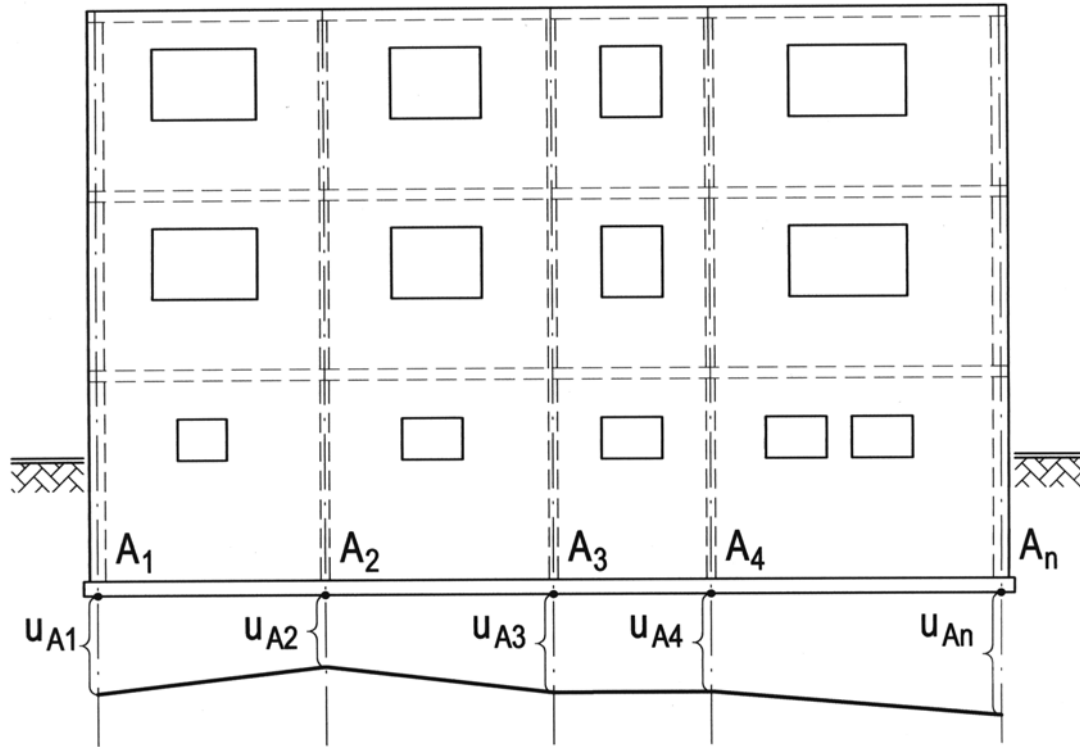


Fig.1 Scheme of soil subsidence of the external load-bearing wall

2.2 Acceptable values of irregular settlements given in Eurocode 7 and other publications

Appendix H of Eurocode 7 (2004) contains admissible values of structure deformations and foundation movements. Definitions of some terms for foundation movement and deformation, which should be taken into account in analysis of structures and buildings, are given in Fig. 2. Generally, the components of movement (mainly in vertical direction) that must be considered in wall analysis include irregular or differential settlement, rotation, tilt, relative deflection, relative rotation and horizontal displacement.

In case of masonry structures (as a matter of fact, made of load-bearing brick walls or part of walls) Eurocode 7 (2003) gives the maximum acceptable relative rotations “(...) to range from about 1/2000 to about 1/300, to prevent the occurrence of a serviceability limit state in the structure. A maximum relative rotation of 1/500 is acceptable for many structures. The relative rotation likely to cause an ultimate limit state is about 1/150.” All these ratios should be applied to a sagging mode (see Fig. 2), whereas for a hogging mode (edge settling more than part between), the adequate values of acceptable in-plane deformations should be halved.

Appendix H of Eurocode 7 (2003) contains also the following additional requirements: “(...) For normal structures with isolated foundations, total settlements up to 50 mm are often acceptable. Larger settlements may be acceptable provided the relative rotations

remain within acceptable limits and provided the total settlements do not cause problems with the services entering the structure, or cause tilting etc. These guidelines concerning limiting settlements apply to normal, routine structures. They should not be applied to buildings or structures, which are out of the ordinary or for which the loading intensity is markedly non-uniform". Like it was discussed earlier this regulation is not much precise. Moreover, which kind of structure can be treated as "normal, routine structure"?

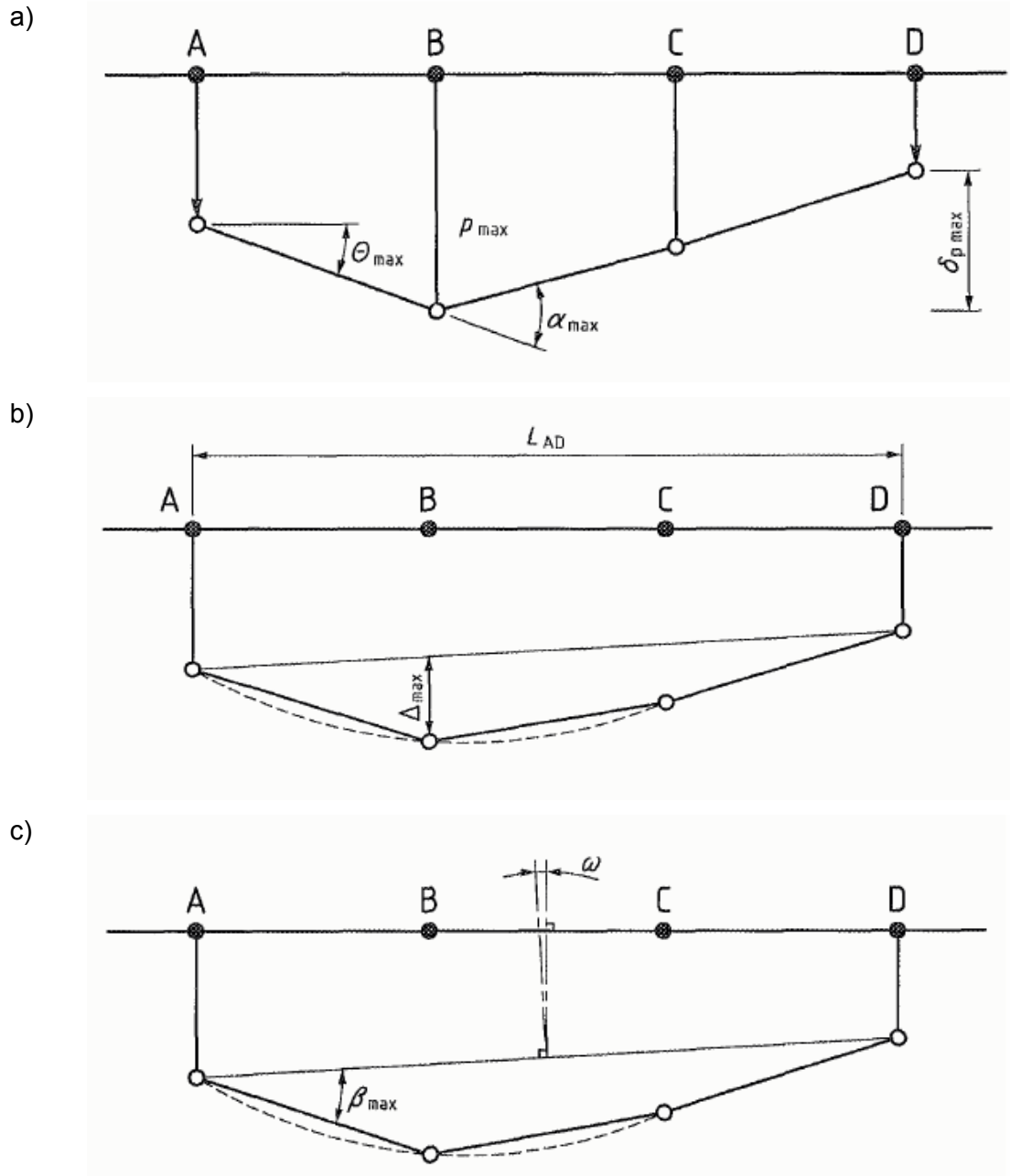


Fig. 2 Definitions of foundation movement – according to Eurocode 7 (2004):

- a) definitions of settlement p , differential settlement δp , rotation and angular strain α ;
- b) definitions of relative deflection Δ and deflection ratio Δ/L ;
- c) definitions of tilt ω and relative rotation (angular distortion) β

Unfortunately, quite similar regulations, like given in Eurocode 7 (2003), can be found in most national standards or other publications. For example the acceptable values of settlements, relative rotations and tilt of buildings taken from still obligatory Polish Geotechnical Code PN-81/B-03020 (1981) are given below in Table 1.

Table 1 Acceptable admissible value of settlement, relative deflection and tilt according to PN-81/B-03020 (1981)

Building type	$\delta_{P,maxr}$ [mm]	ω	Δ_{max} [mm]
Buildings up to 12 stories	7.0	0.003	1.0
Buildings over 12 stories	8.0	0.002	1.0
Slender buildings over 100m high	15.0	0.001	-

In the above Table the notation were used as shown in Fig.2.

Analysis of values from Table 1 shows, that Polish regulations are more restricted than given in Eurocode 7 (2003) for acceptable value of total settlements. For typical buildings (single-storey or not so high multi-storey housing) they should not exceed 7 mm. Moreover, the maximum admissible values of relative deflection (Δ_{max}) are not connected with length of the building or part of wall separated by transverse load-bearing walls. In The relationship between rotation ratio (Δ/L) and different lengths (L) of the wall is shown in Fig.3.

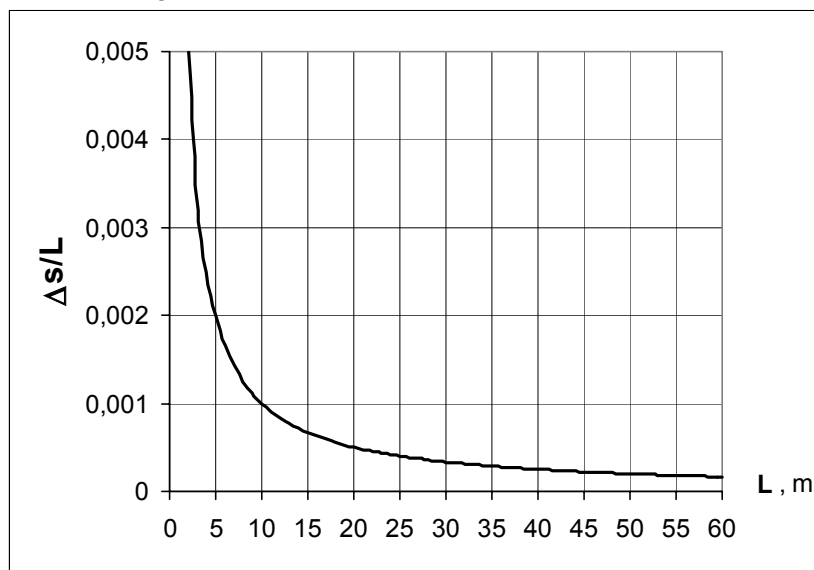


Fig.3 The relationship of rotation ratio (Δ/L) – length (L) of the shear wall

In case of buildings with shear wall length (L) greater than 10,0 m the acceptable value of relative rotation does not exceed 1/1000, whereas for the length of wall (or part of the wall divided by configuration of transverse load-bearing walls) is significantly smaller, e.g. below 5,0 m (what is typical for small up to two-storey buildings with wall load-bearing construction) admissible value of rotation ratio is starting from 1/500 and very quickly growing up for smaller (L) distances. Based on data obtained from large monitoring of a few hundreds of masonry buildings (during the second half of past century) carried out in Poland, Wilun (1982) determined the maximum acceptable value

of rotation ratio (Δ/L) that does not exceed 1/300. This value is less restricted than provided by Eurocode 7 (2003).

It is very wide difference between the both above mentioned maximum acceptable values of relative rotation. Practically, it is not possible for the sake of differences among load bearing configurations of buildings to determine unequivocally values of acceptable settlement differences – even for structures built using similar materials. The same admissible value of relative deflection in one building causes none damages, while in other one can produce damages not acceptable from operation point of view or even can be dangerous. Therefore, it was necessary to look for a method to prevent the occurrence of a serviceability limit state in the masonry structure not only connected with acceptable values of settlements or rotation ratios but based also on strain state of deformed wall. Such method (strain deformation criterion) of analysis was elaborated and introduced into Polish Masonry Code PN-B-03002:1999 (1999).

3 Proposition of a supplement to Eurocode 6

During long discussions of members of Task Committee No 252 (Masonry Structures) of Polish Standard Institution, we decided to prepare a proposition of a supplement to Eurocode 6 (2003) concerning method (accepted and introduced into Polish Masonry Code PN-B-03002:1999 (1999)) of checking the serviceability limit state of masonry wall in-plane deformed as a result of irregular settlements. The proposed method is more universal than the all above discussed regulations based on acceptable values of settlements or rotation ratios. Moreover, one takes also into consideration the type of masonry units and mortar type, that the analysed wall was made of. In our opinion the proposed criterion can be also useful in design practice in other European countries.

Therefore, we suggest to introduce the following changes in section 7.2 of Eurocode 6:

(1) In unreinforced masonry structures, other than shear walls against vertical displacements, Serviceability Limit State for cracking and deflection needs mostly not be checked separately when the Ultimate Limit State has been satisfied.

Note: One should be borne in mind that some cracking could result when the Ultimate Limit State is satisfied, e.g. roofs or floors.

(2) For shear (stiffening) walls against vertical displacements (subjected to vertical shearing – see scheme shown in Fig. 4a) of structures for short-term settlements (immediate during their erection and caused by first part of consolidation) Serviceability Limit State should be checked whether:

$$\Theta_{Sd} \leq \Theta_{Rd} = \frac{\Theta_{Rk}}{\gamma_M} \quad (1)$$

where:

Θ_{Sd} the maximum value of non-dilatational strain angle of most deformed part of analysed shear wall (e.g. determined in calculations using analytical or numerical method),

Θ_{Rk} the characteristic value of non-dilatational strain angle having a prescribed probability of 5% of cases where width of appeared cracks exceed 0,3 mm; is given in Table 2,

γ_M the material partial safety factor (taken according to point 2.4.4 in Eurocode 6 (2003) – in serviceability limit state analysis taken as $\gamma_M = 1$).

Table 2 The characteristic admissible values of Θ_{Rk} (mm/m)

Masonry units	General purpose mortar		Thin layer mortar
	Cement mortar	Cement-lime mortar	
Group 1 excluding Autoclaved Aerated Concrete	0.4	0.5	1)
Group 2,3,4	1)	1)	1)
Autoclaved Aerated Concrete	0.2	0.3	0.2
1) value should be determined by tests			

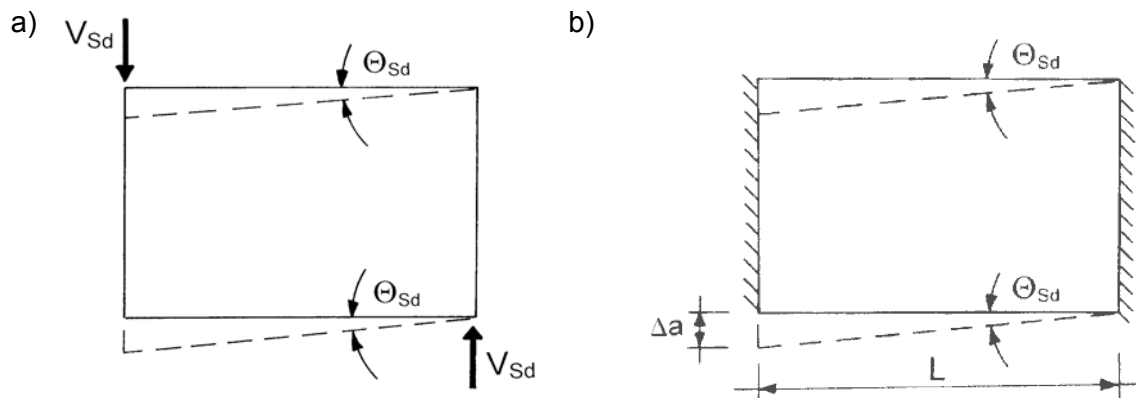


Fig.4 Scheme for non-dilatational strain angles Θ_{Sd} calculation:

- a) acting shear forces;
- b) calculations based on vertical displacement

The values of Θ_{Sd} based on the state of deformation (see Fig.4b) in case of using analytical methods taking into consideration only the values of settlements or vertical displacements of whole or part of the wall, can be determined using the following formula:

$$\Theta_{Sd} = \frac{\Delta a}{L} \quad (2)$$

where:

- Δa difference of settlement (see Fig.4b);
- L distance between two transverse load-bearing walls.

(3) For the state of long-term settlements (including second part of subsoil consolidation and influence of creep) the Serviceability Limit State can be verified as follow:

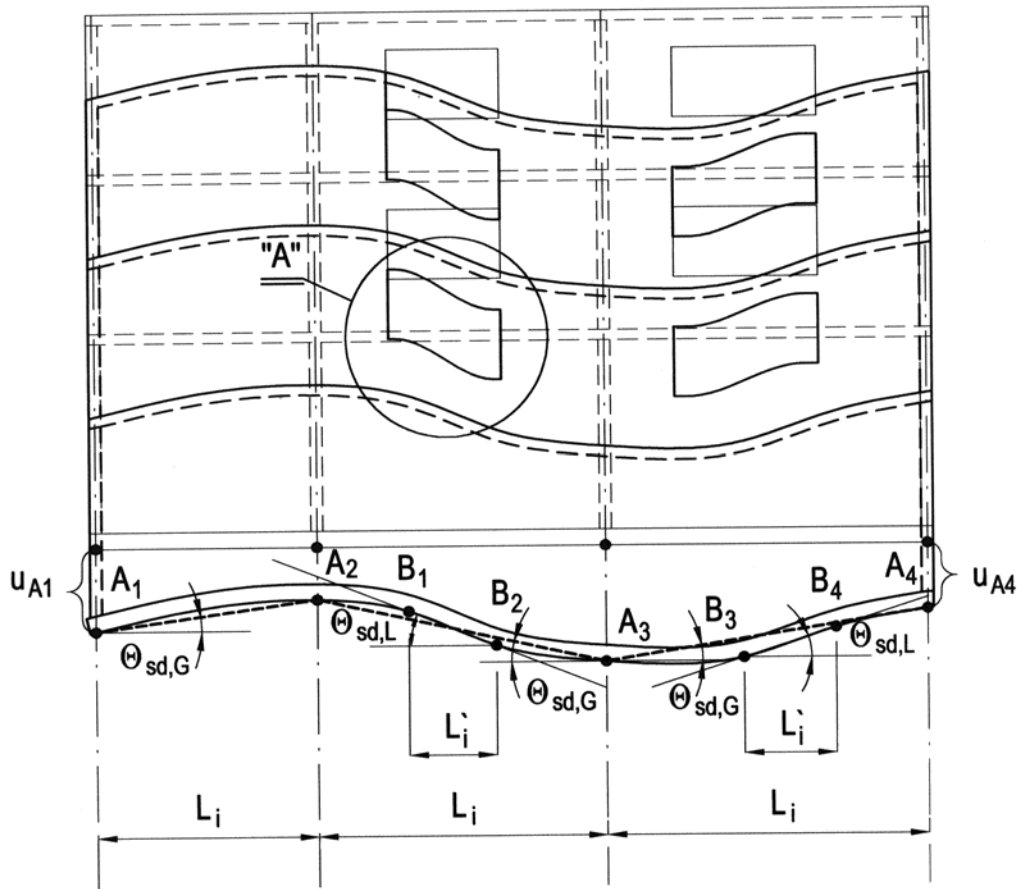
$$\Theta_{Sd} \leq \Theta_{Rd} (1 + \phi_{\infty}) \quad (3)$$

where:

- ϕ_{∞} the final creep coefficient (taken from the table given in point 3.7.4 of Eurocode 6 (2003).

Calculation of design values Θ_{Sd} using formula (2) based only by the differences of vertical displacements between two transversal load-bearing walls is connected with significant simplification because is not taking into account all deformations which

appear along the analysed wall. In real situation, by reason of existing interaction between ground and building and variable stiffness of walls (e.g. part of wall can get openings) the shape of deformed wall is usually complex – see Fig.5.



DETAIL "A"

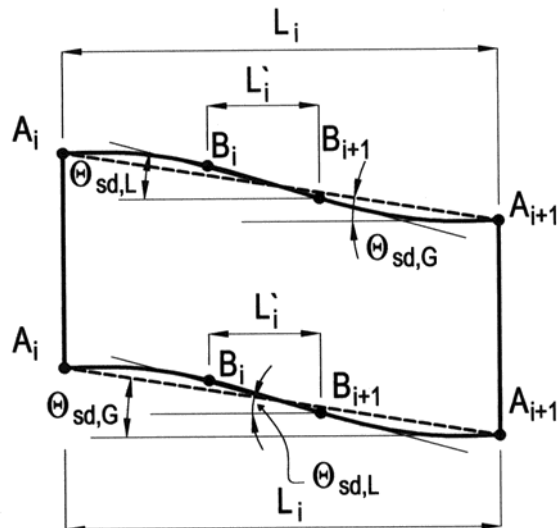


Fig.5 Scheme of deformed masonry wall with openings subjected to irregular settlement

Most part of total vertical displacement differences is observed on segment shorter than the whole length of analysed wall, part of wall (marked in Fig.5 as L_i') or span of window opening (see detail "A" in Fig. 5). In such situation the value of non-dilatational strain angle (marked as $\theta_{Sd,L}$) determined for that shorter segment (L_i') can be much greater than ($\theta_{Sd,G}$) calculated as an average value for the whole length (L).

Taking this fact into consideration the author of this paper made decision to introduce two new notions, namely: global ($\theta_{Sd,G}$) and segmental ($\theta_{Sd,L}$) non-dilatational strain angles defined as follow:

- a) Global non-dilatational strain angle ($\theta_{Sd,G}$) – determined for the whole wall or part of shear wall as:

$$\theta_{Sd,G} = \frac{\Delta u_{A,i}}{L_i} = \frac{|u_{A,i+1} - u_{A,i}|}{L_i} \quad (4)$$

- b) Segmental non-dilatational strain angle ($\theta_{Sd,L}$) – determined for the most deformed part or band of wall (also equal span of window openings) as:

$$\theta_{Sd,L} = \frac{\Delta u_{B,i}}{L_i'} = \frac{|u_{B,i+1} - u_{B,i}|}{L_i'} \quad (5)$$

Notation used in above given formulae in accordance with Fig.5.

4 Summary

The presented method of serviceability limit state verification (several years ago introduced into the Polish Masonry Code PN-B-03002:1999 (1999)) in case of masonry walls subjected to irregular settlements is more effective and, what is important, significantly more precise than all general suggestions and requirements provided in Appendix H of Eurocode 7 (2003) and many other standards and publications. Especially now, when numerical methods of calculations are wide in use, it is not difficult to determine the state of strain (deformation) of analysed wall.

As a correct value in design practice, in author's opinion, the segmental values of non-dilatational strain angle ($\theta_{Sd,L}$) should be taken most frequently. Unfortunately, it is possible only in analysis of masonry walls using numerical method, because only in this case designer can receive the real shape and values of deformation components of the edge of a wall.

The proposed method was verified by test data obtained from investigations carried out, mainly in Department of Building Structures and Bridges of Silesian University of Technology in Gliwice, for last fifteen years.

Taking into account, that using this method of Serviceability Limit State satisfying is equal with satisfying the Ultimate Limit State – the proposed criterion is quite safe and can be useful in design practice.

References

- Building Research Establishment, 1984, Safety of large masonry walls, BRE Digest 281, BRE, Garston.
- Building Research Establishment, 1989a, Simple measuring and monitoring of movement in low-rise buildings: part 1 – cracks, BRE Digest 343, BRE, Garston.
- Building Research Establishment, 1989b, Simple measuring and monitoring of movement in low-rise buildings: part 2 – settlements, have and out-of-plumb, BRE Digest 344, BRE, Garston.

- Building Research Establishment, 1991, Why do building crack?, BRE Digest 361, BRE, Garston.
- Cook D., Ring S., Fichtner W., 1995, The Effective Use of Masonry Reinforcement for Crack Repair, Proc. 4th IMC, Masonry (7), London, United Kingdom, 442-450.
- Freeman T.J., Littlejohn G.S., Driscoll R.M.C., 1994, Has your house got a cracks?, Thomas Telford Ltd, London.
- Greenspan H.F. *et al*, 1980, Guidelines for failure investigation, ASCE, New York, U.S.A.
- Kubica J., 1998a, On Some Indirect Methods of Analysis of Shear Walls, Proc. 5th IMC, Masonry (8), London, United Kingdom, 175-177.
- Kubica J., 1998b, The Polish Approach to the Shear Walls Analysis, Proc. 35th Meeting of CIB/W23 Wall Structural Commission, Dresden 98, Germany, CIB Proceedings, Publication No 242, 89-98.
- Kubica J., 2002, Shear Modulus in Stiffening Walls Analysis – New Polish Standard's Regulations. Proc. 6th IMC, Masonry (9), London, United Kingdom, 253-259.
- Kubica J., 2003, Unreinforced masonry walls subjected to non-dilatational strains produced by irregular vertical ground displacements, Wydawnictwo Politechniki Slaskiej, Gliwice, Poland (in Polish).
- Parkinson G., Shaw G., Beck J.K., Knowles D., 1996, Appraisal & Repair of Masonry. Thomas Telford Ltd, London.
- PN-B-03002:1999, 1999, Unreinforced masonry structures. Analysis and structural design. Polski Komitet Normalizacyjny, Warsaw.
- PN-81/B-0320, 1981, Buildings subsoil's. Direct foundation of buildings. Analysis and structural design, Polski Komitet Normalizacyjny, Warsaw.
- Polshin D.E., Tokar R.A., 1987, Maximum allowable non-uniform settlement of structures, Proc. 4th International Conference of Soil Mechanics and Foundation Engineering, Vol.1, 402-405.
- prEN 1996-1-1, Eurocode 6, 2003, Design of Masonry Structures – Part 1-1: Common rules for reinforced and unreinforced masonry structures.
- prEN 1997-1, Eurocode 7, 2004, Geotechnical design – Part 1: General rules. Final draft.
- Wilun Z., 1982, An outline of geotechnics, Wydawnictwa Komunikacji i Łączności, Second edition, Warsaw (in Polish).