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Evaluation of prediction methods for vertical deformation of GRS walls

Évaluation des méthodes de prévision dela déformation verticale de la GRS murailles

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ABSTRACT: The problem of accurate prediction of deformations of geosynthetic reinforced walls is discussed in this paper. Main attention is focused on an analytical method developed in IBW PAN which can be used for this purpose. It‘s main theoretical assumptions are presented. Two different boundary conditions between geosynthetic and soil are taken into consideration. The first deals with the case when perfect bonding between geosynthetic and soil is assumed. The second – when failure between geosynthetic and soil is possible. Calculation formulaes which can be used for the prediction of GRS deformations under external loading are presented. Their accuracy is compared with experimental results. Main conclusions are presented.

RÉSUMÉ: Le problème de la prédiction précise des déformations des parois géosynthétiques renforcées est abordé dans cet article. Une attention particulière est accordée à la méthode d'analyse développée dans IBW PAN qui peut être utilisée à cette fin. Ses principales hypothèses théoriques sont présentées. Deux conditions limites différentes entre le géosynthétiques et le sol sont prises en considération. La première traite du cas où l'on suppose une liaison parfaite entre le géosynthétique et le sol. La seconde - quand une rupture entre géosynthétique et sol est possible. Les formules de calcul qui peuvent être utilisées pour prédire les déformations de la GRS sous charge externe sont présentées. Leur précision est comparée aux résultats obtenus sur la base de résultats expérimentaux. Les principales conclusions sont présentées.

Keywords: Geosynthetic reinforced soil; bridge abutments; deformations; theoretical methods

1 INTRODUCTION

Geosynthetic reinforced soil (GRS) walls are commonly used in engineering practice. The new technology has a number of advantages including reduced construction time and cost. Current design methods for the GRS walls consider only the stresses and forces in the wall system. There is no commonly accepted design methodology which can be used for the prediction of the GRS wall deformation. The problem of accurately determining strains and displacements in this kind of structures is complex The deformations of the GRS walls depends on many factors. The most significant are: wall height, surcharge loading, facing and reinforcement type and stiffness, backfill soil characteristics, and toe resistance.

According to the commonly accepted opinion the problem of prediction of the reinforced soil walls deformation under applied service loads requires further investigation. It is especcially important in the context of the application of such structures to bridge-supporting structures,

particularly for bridge abutments which are subject to cyclic loading. In these cases the proper monitoring and predicting of deformations under serviceable loading is crucual from the engineering point of view.

The aim of this paper is to present the possibility of using a selected analytical method to calculate the GRS structures deformation. It was proposed and developed by researchers from IBW PAN (Sawicki, 2000; Kazimierowicz-Frankowska, 2003; Kulczykowski, 2013; Kazimierowicz-Frankowska 2018). Basic principles of the proposed methodology are described. Calculation procedures and formulaes obtained for two different conditions on the border between reinforcement and soil are provided. The first deals with the case when perfect bonding between geosynthetic and soil is assumed. The second – when bond-failure is possible. Special attention is focused on the problem of predictiction of the vertical settlements of the GRS structures. This issue especially needs furher investigation because only a few other analytical methods and experimental results are availabe to its solving. They are also discussed within the framework of this paper.

2 SHORT REVIEW OF THE CURRENT STATE OF THE ART

From the practical point of view an engineer is interested mainly in accurately predicting two types of deformations of the GRS structures: horizontal displacements of the facing and vertical displacements (settlements) of the crest – Figure 1. An increasing number of studies on the problem of the GRS horizontal deformations have been conducted by researchers in the last years (e.g. Khosrojerdi et al. 2017, Ozturk 2014, Wu et al. 2013, Bathurst et al. 2009, Kazimierowicz-Frankowska 2018). On the basis of experimental results their character was recognized and described using theoretical approaches – Figure 2.

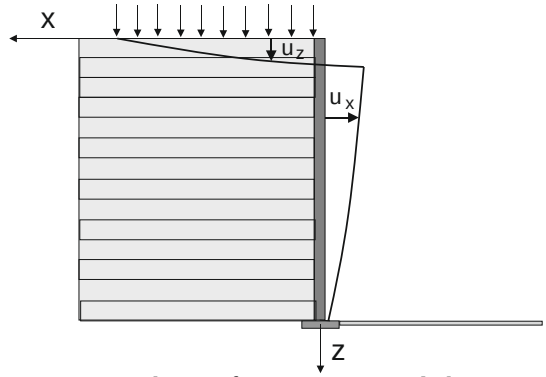
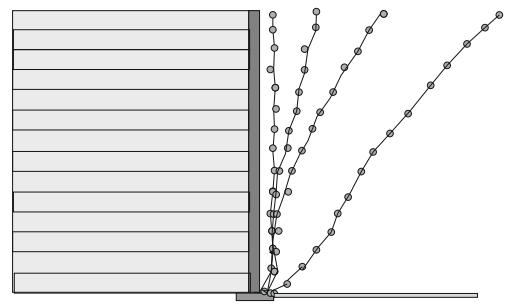


Figure 1 Scheme of GRS structure's behaviour under serviceable loading.

a)



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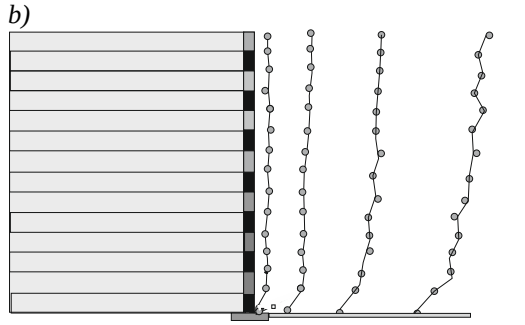

Figure 2 Experimentally determined character of GRS horizontal deformations (Kazimierowicz-Frankowska, 2003): a) concrete panel facing wall; b) segmental facing wall.

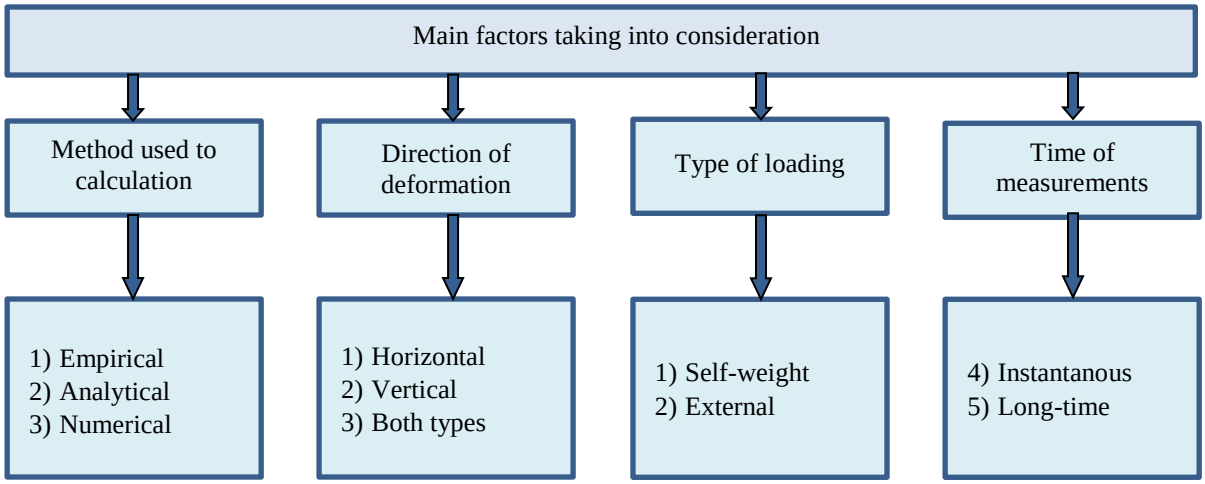

Figure 3 Main factors taking into consideration

Although the progress in the field is clearly visible there is no one commonly accepted method or guideline on how to predict GRS deformations which can be used in their design precedure. Available calculation methods can be classified within the framework of a few groups – Figure 3. All of them have adventages and disadventages (see e.g. Kazimierowicz-Frankowska 2018). This paper focuses on the description and verification of the calculation procedures performed in available analytical methods used for calculation of vertical deformations of the GRS structures. Two methods have been taken into consideration. The first was developed in IBW PAN (Sawicki 2000 and Kazimierowicz-Frankowska 2003). It‘s accuracy was verified on the basis of experimental results performed in both laboratory (Kulczykowski 2013) and full scale experimental tests (Kazimierowicz-Frankowska, 2018). However so far only the investigations of values of horizontal deformations observed under serviceaible loading have been performed. The second method which will be present was proposed by Adams et al., 2014. It was used for example by Nicks et. al., 2016 to estimate the vertical deformations of the GRS abutments.

3 PROPOSED THEORETICAL APPROACH

3.1 Model of GRS wall

The following general assumptions are made in the theoretical approach:

- The active and passive zones of the geosynthetic reinforced walls with cohesionless backfill is considered – see Figure 4.
- A potential mechanism of failure is defined by the planar rupture surface AB, inclined at the angle θ to the horizontal. Two parts of the reinforced zone are selected: the triangular portion ABO, (denoted as the active zone in Figure 4) and the quadrilateral portion ACDB (denoted as the passive zone).
- The soil in the active zone is assumed to be in the plastic state. In this zones perfect bounding between geosynthetic and soil is assumed.
- The wedge ABO remains in the global equilibrium. Adopting some simple assumptions one can determine the distribution of forces in the reinforcement across the potential failure surface AB, corresponding to the global reaction of the reinforcement (Sawicki 2000).

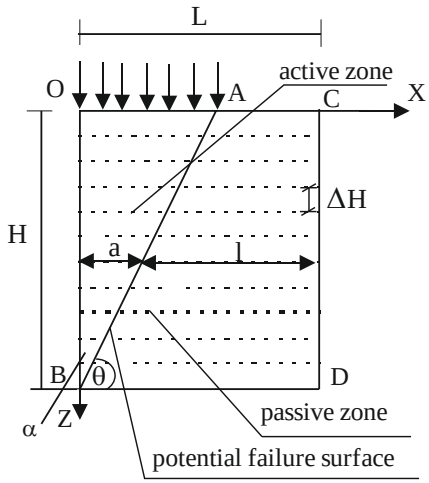

Figure 4 Theoretical model of GRS structure

- The GRS retaining wall is loaded on its upper surface. The load, p, acting on top of the structure is smaller than the collapse load (its lower-bound estimate).
- It is assumed that the horizonal displacement face of the GRS structure consists of two elements:

$$u_x = u_{act} + u_{pass} \tag{1}$$

where u_{act} and u_{pass} are the displacements resulting from the deformation of the reinforcement in respectively active and passive zone.

3.2 Horizontal deformation in the active zone

In the case of reinforcement working in the elastic range, the strain in the active zone is calculated from the Hooke's law:

$$\varepsilon_{act} = F/E \tag{2}$$

where $F = A_r \sigma_X^r$ - force in the reinforcement, $E = A_r E_r$ - elastic stiffness of the reinforcement, A_r - cross sectional area of reinforcement.

The horizontal displacement of the RS facing due to elastic deformation of the reinforcement in the active zone is calculated by the integration of strains:

$$u_{act} = \int_0^{x^*} \varepsilon_{act} dx \tag{3}$$

where

$$x^* = (H - z) tan\,\alpha \tag{4}$$

3.3 Horizontal deformation in the passive zone

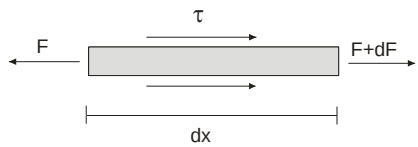

Figure 5. Forces acting on the reinforcement element.

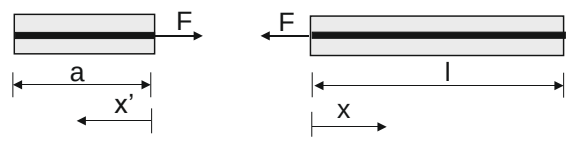

Figure 6. Reinforcing strip in active and passive zones

In order to analyse the forces acting on the reinforcement strip the following four governing equations were used (Sawicki 2000, Kazimierowicz-Frankowska 2003):

- From equilibrium of a single reinforcing element (see Figure 5) the following equation can be written:

$$\frac{dF}{dx} = -2B\tau \tag{5}$$

where F - is the tensile in the reinforcement, B – is the width of the reinforcement strip.

- The strain in the reinforcement strip can be expressed as:

$$\varepsilon = du/dx \tag{6}$$

where u –displacement.

- The linear relationship between sheer stress and the displacement:

$$\tau = -Gu \tag{7}$$

where G is a coefficient of proportionality

- The constitutive relation between tensile strength and strain

$$F = E\varepsilon \tag{8}$$

From equations (5) – (8) one can obtains the following differential equations which describes the distribution of forces along the strip:

$$\frac{d^2F}{dx^2} - \beta^2 F = 0 \tag{9}$$

where $\beta = \sqrt{\frac{2BG}{E}}$.

The solution of the equation (9) is the following function:

$$F = C_1 e^{-\beta x} + C_2 e^{\beta x} \tag{10}$$

where C_1 are C_2 are determined from the respective boundary conditions. For the simplest model of reinforcement pull-out shown in Figure 6 they are following:

$$F = (x=0) = F \text{ and } F = (x=l) = 0 \tag{11}$$

3.4 Vertical deformation

Vertical displacements are associated with the horizontal ones. It was assumed that the Coulomb-Mohr yield condition applies in respect to the soil in the active zone:

$$f_S = \sigma_Z - \sigma_X^S - \left(\sigma_Z + \sigma_X^S\right)\sin\varphi = 0 \tag{12}$$

where σ_X^S means partial stress in soil.

Taking into consideration the associated flow rule one can obtain the following formulas:

$$d\varepsilon_X^{S,pl} = -d\lambda(1+\sin\varphi) \tag{13}$$

$$d\varepsilon_Z^{S,pl} = d\varepsilon_Z^{pl} = d\lambda(1-\sin\varphi) \tag{14}$$

Integration of equations (13)-(14) leads to:

$$\varepsilon_Z^{S,pl} = \int_0^{\lambda} d\varepsilon_Z^{pl} = -\lambda(1+\sin\varphi) \tag{15}$$

$$\varepsilon_Z^{pl} = \varepsilon_Z = \lambda(1-\sin\varphi) \tag{16}$$

Unknown function is given (from the strain compatibility condition) as:

$$\lambda = -\frac{\sigma_X^r}{E_r(1+\sin\varphi)} = -\frac{\sigma_r \Delta H}{E(1+\sin\varphi)} \tag{17}$$

Substituting (17) to (16) one can obtain a formula to calculate the GRS crest strains:

$$\varepsilon_Z = -\frac{\sigma_X^r(1-\sin\varphi)}{E_r(1+\sin\varphi)} = -\frac{\sigma_r \Delta H(1-\sin\varphi)}{E(1+\sin\varphi)} \tag{18}$$

After integration of the strains within the active zone the vertical displacements can be obtained:

$$u_Z = \int_0^{z^*} \varepsilon_Z dz \tag{19}$$

where:

$$z^* = H\left(1 - \frac{x}{H\tan\alpha}\right) \tag{20}$$

4 CALCULATION PROCEDURES AND FORMULAES

4.1 Input values

The following input parameters should be determined before starting the calculations:

- geometrical quantities: height of the RS structure (*H*); length of the reinforcement (*L*); vertical (ΔH) and horizontal (ΔB) spacing of the reinforcement; width of the reinforcement strip (*B*);
- material characteristics: tensile strength of the reinforcement (*R*); stiffness of the reinforcement (*E*); coefficient of friction between the soil and the reinforcement (μ); unit weight of soil (γ); angle of internal friction of soil (φ);
- acting forces: type and magnitude of external load (*p*).

4.2 Case 1: Perfect bonding between the soil and the reinforcement

In the case when perfect bonding between the soil and the reinforcement layers is assumed, the horizontal displacement of facing is caused only by deformation in the active zone. It can be calculated from the following formula:

$$\varepsilon_x^a == \frac{\Delta H(p+\gamma z)\tan^2\alpha}{E} \tag{21}$$

Substituting equations (4) and (21) to (3) the following forlulae for horizontal displacement of the GRS wall can be obtained:

$$u_{act} = \frac{\Delta H \tan^3\alpha}{E}(p+\gamma z)(H-z) = \\ = -\xi\left[\gamma z^2 + (p-\gamma H)z - pH\right] \tag{22}$$

where: $\xi = \frac{\tan^3\alpha}{E}\Delta H$.

Vertical strain are calculated from the equation (18). Substituting to (18) the formula:

$$\sigma_z = (p+\gamma z)\tan^2\alpha \tag{23}$$

the following final relationship is obtained:

$$\varepsilon_z = \theta^*(p+\gamma z) \tag{24}$$

where:

$$\theta^* = \frac{\Delta H(1-\sin\varphi)\tan^2\alpha}{E(1+\sin\varphi)} \tag{25}$$

Substituting equations (25) and (20) to (19) the vertical displacement can be calculated as:

$$u_z = \theta^* H\left(1-\frac{x}{H\tan\alpha}\right)\left[\begin{array}{l} p+\frac{1}{2}\gamma H \\ \left(1-\frac{x}{H\tan\alpha}\right)\end{array}\right] = \\ = \theta^* H\left[\begin{array}{l}\left(p+\frac{1}{2}\gamma H\right) - \frac{1}{\tan\alpha}\left(\frac{p}{H}+\gamma\right)x + \\ + \frac{\gamma}{2H\tan^2\alpha}x^2\end{array}\right] \tag{26}$$

4.3 Case 2: Bond-failure between geosynthetic and soil in passive zone

In the second case, when bond-failure between reinforcement and soil in passive zone is possible, firstly the depth of the pull-out zone should be determined. More information about this subject is presented by Sawicki (2000). Here, it was assumed that calculations will be performed for the value of z which is in the range when pull-out is possible (within the pull-out zone). In this case the maximum horizontal displacement (u_x) of the GRS structure's facing consists of two parts (displacements in the active and passive zones). Due to lack of space in this paper only final formulas will be presented for this case. They are a little more complicated than in the previous one (when perfect bonding was assumed), however in general the calculation

procedure is easy and fast for users with personal computers.

The displacement of GRS in active zone can be determined from the following equation:

$$u_{act} = \frac{\omega \Delta H \tan\alpha}{E} \cdot \left\{ \begin{array}{l} pH(L - H\tan\alpha) + \left[\begin{array}{l} p(2H\tan\alpha - L) + \\ \gamma H(L - H\tan\alpha) \end{array} \right] z - \\ \left[p\tan\alpha + \gamma(L - 2H\tan\alpha) \right] z^2 - z^3 \gamma \tan\alpha \end{array} \right\} \quad (27)$$

The second one (horizontal displacement in passive zone) is obtained from:

$$u_{pass} = \frac{\sigma_r \left(1 + e^{-2\beta l}\right)}{\beta E \left(1 - e^{-2\beta B l}\right)} \Delta H \quad (28)$$

where $\sigma_r = \xi \left[\gamma (L - h\tan\alpha) z + \gamma z^2 \tan\alpha \right]$, l -length of reinforcement in passive zone, $\beta = \sqrt{\frac{2BG}{E}}$ = coefficient, G – coefficient (interface stiffness) which is determined experimentally; usually in the range between 10^3 and 10^4 kN/m^3

Vertical displacement of GRS crest are calculated using (18). In this case it leads to the following final formula:

$$u_z = \int_0^{z^*} \theta^* \xi \left(A^* + B^* z + C^* z^2 \right) dz \quad (29)$$

Simple mathematical manipulations lead to the following result:

$$u_z = \Theta^* \xi \left[\begin{array}{l} H\left(A^* + \frac{1}{2} B^* H + \frac{1}{3} C^* H^2 \right) - \\ - \frac{1}{\tan\alpha} \left(A^* + B^* H + C^* H^2 \right) x + \\ \frac{1}{\tan^2\alpha} \left(B^* + C^* H \right) x^2 - \\ \frac{C^*}{3\tan^3\alpha} x^3 \end{array} \right] \quad (30)$$

where:

$$A^* = p(L - H\tan\alpha) \quad (31)$$

$$B^* = \left[(p - \gamma H)\tan\alpha + \gamma L \right] \quad (32)$$

$$C^* = \gamma \tan\alpha \quad (33)$$

5 THEORY VERSUS EXPERIMENT

5.1 Experimental tests

The accuracy of the proposed analytical approach was verified on the basis of selected experimental results taken from literature. The most significant information concerning input parameters is presented in Table 1. Both types of cited experimental investigations deal with the examination of behaviour of the GRS structures used as part of bridge systems. The first (Nicks et al, 2016) evaluates the results of large scale GRS column load tests in order to study the effect of tensile strength, vertical reinforcement spacing, facing elements, and backfill properties on the deformations of the GRS at 200 kPa (typical bridge bearing pressures). The second (Saghebfar et al., 2017) deals with the monitoring of the GRS integrated bridge system.

5.2 Comparison of experimental and theoretical results

Table 1. Experimental data obtained for p=200kPa

Reference	**Basic input parameters**
[1] Nicks et al. 8-70-8-B	H=2.0m;L=1,0m; ΔH=0.194m; R=70kN/m;
[1] Nicks et al. A1a-52-12-B	H=2.0m;L=1,0m; ΔH=0.286m; R=52kN/m;
[1] Nicks et al. A1a-20-4-B	H=2.0m;L=1,0m; ΔH=0.097m; R=20kN/m;

Results of comparison of the experimentally and theoretically obtained data is presented in Table 2. The last set of values were given for the assuption of perfect-bonding between the soil and the reinforcement.

Table 2. Comparison of the theoretical and experimental results of vertical displacements [mm]

Reference	Experiment	Calculation
[1] 8-70-8-B	13.4	17.09
[1] A1a-52-12-B	19.3	19.60
[1] A1a-20-4-B	10.3	11.18

6 CONCLUSIONS

The main purpose of this paper was to present the basic principles of the analytical method which can be used to estimate deformations of the GRS structures under serviceable conditions. Special attention was paid to the problem of accurate prediction of vertical displcements of such structures as geosynthetic reinforced soil integrated bridge systems and GRS abutments.

Calculation procedures are presented for two different conditions on the border between the reinforcement and the soil. The first deals with the case when perfect bonding is assumed. In the second one - bond-failure of the upper reinforcement layers is possible. Theoretical values of vertical displacements were compared with experimental results obtained while monitoring the behaviour of selected structures. Athough the first results of their verification are promising, additional research is needed to verify the accuracy of the proposed methodology for a wider range of GRS structures. The proposed approach, in the author‘s opinion may be an alternative to the commonly used numerical methods.

7 REFERENCES

Bathurst, R.J., Nernheim, A., Walters, D.L., Allen, T.M., Burgess, P., Saunders, D.D. 2009. Influence of reinforcement stiffness and compaction on the performance of four geosynthetic reinforced soil walls, *Geosynthetics International*

Kazimierowicz-Frankowska, K. 2003. Deformations of reinforced soil retaining walls, *Geosynthetics International*, 10(5), 153-164.

Kazimierowicz-Frankowska, K. 2018. Deformations of reinforced soil retaining walls, *Proceeedings of 11th International Conference on Geosynthetics*, Seul.

Khosrojerdi, M., Xiao, M., Qiu, T., Nicks, J., 2017. Evaluation of prediction methods for lateral deformation of GRS walls and abutments, *Journal of Geotechnical and Geoenvironmental Engineering*, 143 (2),

Kulczykowski, M. 2013. Theoretical analysis of RS structures deformation. Internal report. IBW PAN.

Nicks, J.E., Esmaili, D., Adams, M.T. 2016. Deformations of geosynthetic reinforced soil under bridge service loads, *Geotextiles and Geomembranes*, 44, 641-653.

Ozturk, T.E. 2014. Artificial neural networks approach for earthquake deformation determination of geosynthetic reinforced retaining walls, Int. Journal of Intelligent System and Applications in Engineering, 2 (1), 1-9.

Sawicki, A. 2000. Mechanics of reinforced soil. A.A. Balkema/ Rotterdam/ Brookfield, Netherlands.

Saghebfar, M., Abu-Farsakh, M., Ardah, A., Chen Q. 2017. Performance monitoring of geosynthetic reinforced soil integrated bridge system in Louisiana, *Geotextiles and Geomembranes*, 45, 34-47.

Wu, J.T., Pham, T.Q., Adams, M.T. 2013. Composite behaviour of geosynthetic reinforced soil mass. Report No. FHWA-HRT-10-077, McLean, VA.