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The role of junction strength in the performance of geogrids

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ABSTRACT
Designing the performance of geogrids in reinforce soil constructions usually does not consider long-term behavior and possible failure modes of junctions between longitudinal and transverse ribs. How could behavior of junctions be included? To which extend is it necessary? On the conference Geo-Chicago 2016, Swan and Yuan presented an ultimate limit state equation for the short-term material resistance of junctions. We discuss limitations and suggest improvements to include long-term behavior of junctions. Nevertheless, the approach applies only to a specific mode of shear-tensile failure of junctions and only to strictly rigid geogrids. A new design concept presented in the same year by Jacobs on the conference GeoAmerica for the special case of the anchorage of geogrids, which reinforce capping system on long and steep slopes of landfills, tried to overcome these drawbacks. We discuss the aspects of this concept related to the long-term behavior of junctions and the interplay between the load applied to junctions and the flexibility of longitudinal ribs. This interplay and the long-term junction strength determine the long-term behavior of geogrids.

1. INTRODUCTION

Reinforced structures with geogrids (GGR) – e.g. a GGR-reinforced capping system on long and steep slopes of a landfill, where the GGR prevents sliding and is anchored on top of the slope in an anchor trench, or a GGR-reinforced retaining structure to buttress the toe of such slopes – can fail because internal shear failure of the soil occurs, the longitudinal or transverse ribs tear and the junctions between these ribs tear, peel or shear off. Accordingly, an ultimate limit state equation must consider three material resistances: shear strength of soil, tensile strength of ribs and strength of junctions. The extent to which these resistances are mobilized along the GGR depends on – even in case of the ultimate limit state – how stiff the longitudinal ribs are. (In this paper the terms flexible and stiff or rigid refer to the stress-strain behavior of longitudinal ribs and not to the flexural rigidity of the entire GGR).

However, these various aspects of the behavior of GGR are often simplified by tacitly assuming that the GGR is rigid, and the junctions are indestructible (Bräu et al. 2011). Till today it is unclear when and to what extent such a simplified description of the behavior is appropriate (Müller 2011, 2014). In fact, experiences with GGR in reinforced structures have been overwhelmingly positive (Koerner 2012). However, the problems due to simplifications on the side of the evaluation of resistances could have been veiled because either the construction has been carried out with high safety on the side of the load or other resistances have been underrated (e.g. deviation forces at top or toe of an anchor trench).

In the 1990s the relevance of the two topics – limited strength of junctions and limited stiffness of longitudinal ribs – for the behavior of GGR were intensively discussed (Wilson-Fahmy et al. 1993, 1994). One of the practical implications was: "Considering the fact that a great portion of the pullout force may be transmitted by transverse ribs to the junctions … the long-term resistance of junctions should be challenged in determining the anchorage capacity of geogrids". Nevertheless, this topic was set aside in the years to come. The second topic was taken up again in the early 2000s, initially without great response (Ziegler et al. 2004). A similar approach was suggested by Ezzein et al. (2015) and Bathurst et al. (2016). The effects of the flexibility of the longitudinal ribs and the limited junction strength on the anchorage were discussed by Müller (2011, 2014).

Recently, a dissertation was presented by Jacobs (2016) dealing with these issues (Jacobs et al. 2015, 2016). A new design concept for anchor trenches of the GGR of GGR-reinforced landfill capping systems was developed. The concept was based on the simulation of the GGR by a discrete element model (Ziegler et al. 2004, Müller 2011, 2014), which is in fact equivalent to the incremental load finite element model used by Wilson-Fahmy et al. (1993, 1994). (Besides, even though a difference in the properties of transverse and longitudinal ribs can be included into such models, this is not considered here. Mechanical and degradation behavior of transverse ribs and longitudinal ribs of many GGR are equal and can be treated in the same way).The discrete element model was, however, modified to include the effect of the geometry of the anchor trench on the anchorage resistance and to allow for different failure modes of the anchorage. The effect of the geometry was taken into account by an appropriate variation of the normal stress along the embedded GGR. At each step of the simulation it was checked whether sliding or pull-out of the GGR is more critical. Hence, this concept is able to consider all relevant aspects, namely the limited stiffness and strength of the ribs, the limited strength and different failure modes of the junctions, the specific geometry of the anchor trench and the possibilities of different failure modes of the anchorage (pull-out and sliding of the GGR). Jacobs (2016) concentrated on the difference between a conventional and his description of GGR behavior with regard to flexibility of the longitudinal ribs and anchor trench geometry. This paper shortly discusses the results in section 5.3.

However, the focus in the following sections is on the effects of the limited strength of junctions and its degradation. In (Müller 2011, 2014), it was already suggested to include limited junction strength into the discrete element model of GGR-soil interaction by a cut-off value for the range of interlocking forces. The interlocking force results from the resistance against pull-out or sliding due to the interlocking of soil particles in the opening of the GGR. This force can only fully evolve, if the junction can sustain it. Jacobs (2016) discovered in his dissertation that, in addition to a cut-off value for the range of the interlocking force because of limited junction strength with respect to tensile-shear stress, a cut-off value for the displacement of the transverse ribs might be necessary to allow for the effect of peeling of the junctions (Jacobs 2016).

Parallel to and independently of this, the relevance of junction strength for the design of reinforced structures was taken up in a contribution to the conference Geo-Chicago 2016 (Swan et al. 2016). Their paper considered how junction strength should be included into the description of GGR behavior. The maximal possible interlocking force for a given GGR and soil was calculated by the formulas based on an analogy between strip foundation and transverse rib and compared with experimentally determined tensile-shear strength of the junctions of the GGR. Thereby, they stuck to the assumption of a completely stiff GGR.

In the following, these developments are discussed, limitations described and improvements proposed. For this, the interaction between soil and GGR (friction and interlocking) and the effect of flexibility of longitudinal ribs is briefly considered. The paper analyzes the limitations of the concept of Swan et al. (2016) and discusses degradation of junction strength by aging processes. The model of Jacobs (2016) and Jacobs et al. (2015, 2016) is briefly introduced, which describes the interplay of the flexibility of longitudinal ribs and loading of junctions, and the handling of the peeling strength of junctions is discussed. Finally, the behavior of GGR with limited stiffness and limited junction strength is described. Hence, this paper tries to bring together the different viewpoints. Thereby, more insight can be gained into which specific properties are relevant for an appropriate description of a GGR.

2. INTERACTION OF SOIL AND GGR

Forces are transmitted between GGR and soil by two mechanisms (Lopez 2002): 1. Interface friction (including possibly adhesion) between soil particles and the surface of the GGR. 2. Interlocking of soil particles in the openings of the GGR and bearing resistance of the soil. The soil “bears” on the transverse rib like it bears along a strip foundation.

To which extent local bearing resistance is mobilized depends on local displacement of the GGR. The interlocking force due to the bearing resistance of the soil is transmitted to the longitudinal ribs via the junctions. The resistance against pull-out is due to interface friction and bearing capacity of the soil.

The conventional approach to the description of GGR behavior relies on the approximations that the GGR is rigid, i.e. displacement is the same everywhere, and that the junctions are indestructible. Hence, local friction force and local interlocking force do not vary along the GGR. Then, the overall interface frictional force and overall interlocking force are proportional to shear strength of the soil, anchorage length L and normal stress σ_n and one obtains the following pull-out resistance R in case of a cohesionless backfill soil (φ: soil friction angle, λ: interaction coefficient, λ_F: interface friction coefficient, λ_I: interlocking coefficient) (Bräu et al. 2011, Lopez 2002):

$$R = \lambda \tan\varphi \, L \, \sigma_n = (\lambda_F + \lambda_I) \tan\varphi \, L \, \sigma_n \qquad [1]$$

In case of flexible GGR with limited strength of the junctions, the mechanisms are locally the same. However, since displacement varies along the GGR, overall behavior is different (Ziegler et al. 2004, Sieira et al. 2009, Müller 2011, Jacobs et al. 2015, 2016, Bathurst et al. 2016).

There are GGR where the interface friction dominates the pull-out resistance to such an extent that the GGR behavior is dominated by this mechanism (friction dominant GGR). If static interface friction is fully mobilized even at very small displacements and is larger than sliding interface friction, Eq. 1 should be applicable to such friction dominant GGR without restriction, even if the longitudinal ribs are highly flexible. In addition, if $\lambda = \lambda_F$ and λ_F is taken from pull-out tests using only longitudinal ribs without transversal ribs, behavior of junctions becomes irrelevant.

For most products, interface friction is small, and the interlocking effect and the shear strength of the soil determine essentially the pull-out force (interlocking dominant GGR). If the flexible longitudinal ribs of the entrenched GGR are loaded by a tensile force, the displacements of the transverse ribs are not evenly distributed. The tensile force is largest in the front area of the embedding and decreases along the longitudinal ribs. It continuously decreases due to the already mobilized interface friction and interlocking forces. Because of the decreasing tensile force, the displacement will decrease as well. From a certain critical length onward, there is no (or only a too small) displacement and no friction and interlocking resistance is mobilized. At a certain value of the tensile force, either the soil or the junctions in the front area of the embedding fail by internal shear failure or junction rupture. The soil fails once its bearing capacity is reached. The junctions rupture, if they cannot sustain the forces, which must be transferred from transverse to longitudinal ribs at or below the bearing capacity of the soil. Failure of soil or junction must happen at first in the front area because displacement and accordingly mobilized bearing resistance is largest there. If the tensile force is further increased the "zone" of soil failure or junction rupture will move along the GGR. The process is like the opening of a zipper. Having reached the end, pull-out of the entire GGR will start. Hence, the maximum tensile force (pull-out force) occurs in between of a succession of local limit states. The assumption inherent to the conventional approach that the pull-out force would increase linearly with the embedded length, is therefore not applicable (or only an approximation of unknown accuracy) for a flexible interlocking dominant GGR. The strength of the junction in the long run is as important for the behavior of the GGR as that of the longitudinal ribs.

3. MAXIMUM FORCE AT JUNCTIONS

At the bearing capacity, the soil is driven into a readily deformable state and the transverse rib can plow through it without further increase of the pulling tensile force (if the ribs or the junctions do not rupture). Jewell (1996) proposed to estimate the related maximum interlocking force using the soil mechanical theory of the bearing capacity under a strip foundation (Figure 1 and Figure 2). The maximum force F_1, which the transverse rib can exert on the soil according to this analogy, is given by (d: thickness of the transverse rib, b: length of the transverse rib (equal to width of the opening):

$$F_1 = bd\,\sigma_n \tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right) e^{\pi \tan\varphi} \qquad [2]$$

The analogy is crude. The soil mechanical theory, which takes the soil to be a continuous and homogeneous elastic continuum, is applied to the embedded GGR even though the backfill material it not such a continuum. It does certainly not consist of a fine and dry powder, the grains of which having a diameter much smaller than the thickness of the transverse rib. The curves of sliding in case of a general shear failure below a foundation (Fig. 2) are taken to be representative, even though the deformation of the soil in case of a GGR leads from the front of the transverse rib to the backside. The pressure p acting on a soil to the left and right of the foundation is identified with σ_n, even though σ_n acts perpendicular to the transverse rib (Figure 1) while in case of a foundation the pressure p acts parallel the force which the foundation exerts on the soil (Figure 2), i.e. it is assumed that the coefficient of earth pressure at rest is 1. However, data from measurements of the pull-out resistance of rigid grids made of steel have shown that Eq. 2 provides an upper limit for the range of measured values (Jewell 1996). Therefore, this formula should be considered as an empirical relation motivated by soil-mechanical considerations, which however must be empirically justified.

When a pull-out of the entire GGR is enforced, the entire "package" of gravel or sand particles, which are jammed into the openings of the GGR, is displaced. Two shear surfaces occur, which lie above and below the plane of the GGR in the backfill itself. Jewell (1996) called it the fully rough state. If the opening of the GGR has length a and width b, the maximum force acting on a transverse rib due to the shear strength of the soil is in the fully rough state (Jewell 1996):

$$F_2 = 2\,ab\,\sigma_n \tan\varphi \qquad [3]$$

According to Jewell (1996), the increase of the force F_1 with shear strength of the soil and normal stress should be limited by F_2, because as soon as the local interlocking force F_1 becomes larger than interlocking force of a fully rough state F_2, it should be more favorable to cross over to the fully rough state. This argument is, as will be discussed in section 4.2.1, not convincing.

4. A LIMIT STATE EQUATION FOR JUNCTIONS

4.1 The concept of Swan et al. (2016)

The authors suggested the following approach. For a cohesionless soil with soil friction angle φ and for a normal stress σ_n, the maximum possible force F having to be transmitted to the longitudinal ribs via a junction is calculated. The ultimate limit state equation compares F with the tensile-shear strength of the junction T_0 as measured in the laboratory (see section 4.2.2).

F is the sum of three components:

1. F_1 and F_2 are calculated according to Eq. 2 and 3 and the respective smaller force $F_3 = \min(F_1,F_2)$ is used.

2. The interface friction force F_{IF} between the GGR and the soil particles is, as usual (Koerner 2012), calculated according to: $F_{IF} = 2\,wb\,(\sigma_n \tan\delta + c)$, where w: width of transverse rib in longitudinal direction, δ: soil-GGR interface friction angle, c: adhesion at soil-GGR interface.
3. On the back side of the transverse rib, in the view of Swan et al. (2016), there is an additional pushing force similar to the force due to the active earth pressure exerted by the backfill on a yielding retaining wall: $F_H = bd\,\sigma_n/\tan^2(\pi/4+\varphi/2)$.
Hence:

$$F = F_3 + F_{IF} - F_H \qquad [4]$$

For given φ, σ_n and GGR properties a, b, d, w, δ and c, the ultimate limit state equation with the global safety factor γ is:

$$T_0 = \gamma\, F \qquad [5]$$

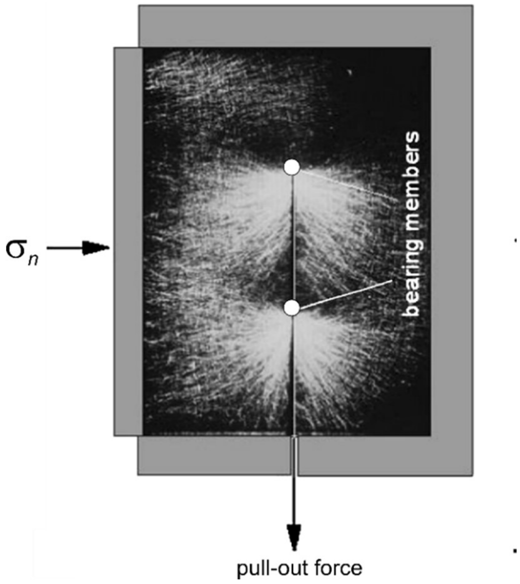

Figure 1. Distribution of stress in front of a transverse rib (bearing member) made visible by photo-elastic methods in a small-scale pull-out test using crashed glass (from Dyer 1985, Palmeira 2004, 2009).

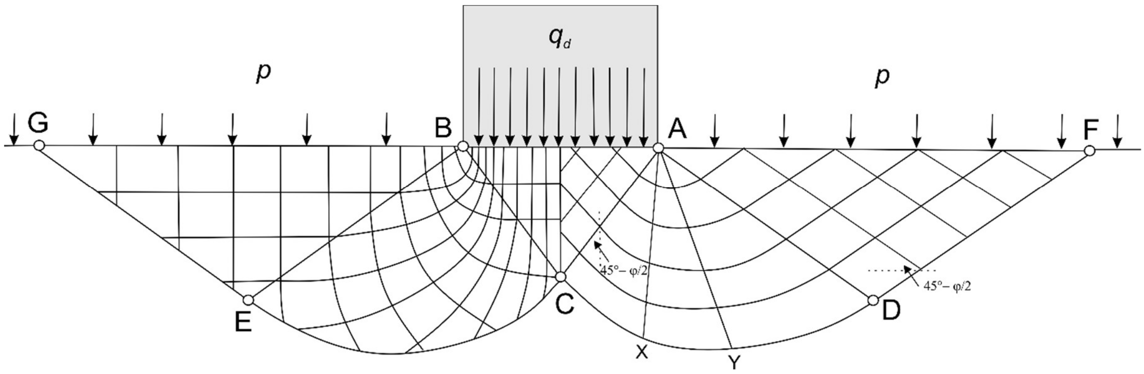

Figure 2. (From Prandtl, 1920). Bearing capacity of a weight- and cohesionless soil q_d depends on the surcharge p. At failure, the soil yields along the curves of sliding (right half). The foundation readily sinks into the ground (general shear failure). If the loading stress due to the strip foundation is $q_d = p\tan^2\left(\frac{\pi}{4}+\frac{\varphi}{2}\right)e^{\pi\tan\varphi}$, the soil will fail (Prandtl 1920, 1921, Reissner 1924). Left half: trajectories of the principle stresses. Weightless means that the stresses due to the weight of the soil are small compared to p and q_d.

4.2 Discussion of the limit state equation

The suggestion of Swan et al. (2016) is a step forward in handling the long pending problem of how to include the junction strength into the assessment and the description of the behavior of GGR. However, the authors of this paper think that essential modifications are necessary.

4.2.1 The theoretical concept

The concept sticks to the theoretical idea of a rigid GGR, because it is assumed that if locally F_1 becomes as large as F_2, there is a cross over to a fully rough state and F_3 is taken to be F_2 instead of F_1. However, this assumption only applies, if all transverse ribs are loaded in the same way by a pulling tensile force, i.e. in the case of a rigid GGR. Theoretical considerations as well as carefully analyzed experiments have clearly shown that in the case of polymeric GGR this assumption is not valid (Ziegler et al. 2004, Sieira et al. 2009, Müller 2011, Jacobs et al. 2015, 2016, Ezzein et al. 2015, Bathurst et al. 2016). The displacement of the transverse ribs and thus the mobilization of the soil resistance is not constant along the length of the embedded GGR. Part of the length does not even contribute. Hence, F_1 can become locally larger than F_2 because the cross over to a fully rough state involving the entire GGR is not at once possible. (Besides, the force F_H is already contained in the force F_1, if Eq. 2 is taken as an empirical relation justified by small scale pull-out tests).

4.2.2 Durability of junctions

The bearing resistance of the soil leads to tensile and shear forces within the plane of the junction and possibly to a related tensile-shear failure. The strength of the junction T_0 with respect to this failure mode is used in Eq. 5. It can be measured either by a test of individual junction strength standardized as method a of ASTM D 7737 or as method b of ASTM D 7737, which gives the unconfined or confined tensile-shear strength of the junction T_0, respectively (Koerner 2012, Kubec et al. 2004, 2004b). However, just as in the evaluation of the design value of the tensile strength of the longitudinal ribs, the laboratory value of the strength of the junction must be modified by various reduction factors to obtain a value relevant for design. Junctions are the weak points of a GGR considering degradation. This applies to woven, knitted, extruded or welded GGR. Like the longitudinal ribs, junctions are subject to aging and various environmental influences (reduction factor R_{CH} and possibly R_W for weathering effects according to ISO/TR 20432). Creep can influence junctions (reduction factor R_{CR} according to ISO/TR 20432). Just as ribs, junctions can be damaged during installation (reduction factor R_{ID} according to ISO/TR 20432). Finally, dynamic loads can have an adverse effect on junction strength (this factor is designated by R_{DL}). If ribs are exposed to all these effects, the junctions are as well. Such reduction factors quantify the loss of strength, which unavoidable takes place during installation and use. It is therefore necessary to insert a design value T into Eq. 5, which is reduced compared to the test value T_0:

$$T = T_0 / (R_{CR} \times R_{ID} \times R_{CH} \times R_{DL}) \qquad [6]$$

If degradation affects junctions in the same way as ribs, the reduction factors of ribs might be used in Eq. 6. However, there are examples where this is not guaranteed. For example, PET material in a weld seam could be so pre-damaged by the thermal and mechanical stress during welding that a more rapid hydrolytic aging occurs. Therefore, a test should be undertaken, in which the hydrolytic degradation of junction strength is compared with that of tensile strength of the longitudinal rib under the same conditions. Another example is when junctions fail by mechanisms, which do not occur in the ribs. Extruded GGR are often made of partially crystalline polyolefins. The ribs are highly stretched to achieve high strength. Molecules and crystallites are strongly oriented and thereby stress cracking is hindered. This does not apply to the junction area between the ribs, where poorly oriented material is exposed to stress concentration. Therefore, checking the stress cracking behavior of junctions under the stress level of the field application is necessary to understand whether stress cracking is an issue during the lifetime of the project. In such cases, one can conduct ASTM D 7737 tests on aged samples or long-term tensile load tests on the junctions of the GGR.

4.2.3 Failure of junctions

In extruded GGR, the transverse rib, the longitudinal rib and the junction lie in the same plane (integral junction). In welded or woven junctions, the transverse and longitudinal ribs lie one above the other. Ziegler and Timmers (2004) demonstrated that in this case the transverse ribs rotate to some extend during the displacement in the soil by using steel grids, which incur irreversible plastic deformation during pull-out. The welded PET GGR excavated after pull-out testing showed similarly deformed transverse ribs. Hence, the bearing resistance of the soil will not only lead to tensile and shear forces within the plane of the junction and possibly to a related tensile-shear failure. In addition, a pair of tensile forces perpendicular to the area of contact between longitudinal and transverse ribs will develop, which will try to separate the junction and possibly leading to a peeling failure (Kubec et al. 2004, 2004b). Welded seams or adhesive bonded junctions are sensitive to peeling forces. Peeling strength is usually much lower than tensile-shear strength. Jacobs (2016) observed that the tensile load, at which the welded junctions failed in the soil, was lower than their confined tensile-shear strength and attributed this to peeling effects. The occurrence of peeling failure depends on the amount of twisting of the transverse rib, which increases with the amount of displacement. In addition to a limit state equation related to a laboratory value of the tensile-shear strength, a criterion for the permissible displacement should be established. In the concept of Jacobs (2016) such a criterion is used, see section 5.2.

4.2.4 Example

Swan et al. (2016) gave an example: a GGR in a supporting layer (φ = 37 °) under a roadway. With σ_n = 77 kPa from traffic load, F_1 was 169 N. A fully rough state, on the other hand, gave F_2 = 75 N. The interface friction F_{IF} of the chosen GGR product and the active earth pressure F_H provided only neglectable contributions. Therefore, F was 75 N. Laboratory tensile-shear strength was T_0 = 245 N. The global safety was then γ = 3.2. According to the arguments in section 4.3 this calculation should be modified as follows. The interlocking force can locally become significantly greater than 75 N, since the tensile force is unevenly distributed along the embedded GGR. Assuming, for example, that all reduction factors multiply to a value of 2 (see, for example, BAM-certified GGR (BAM 2013)), the factor of safety would only be γ = 1 at a value of F = F_1 = 120 N, i.e. long-term strength of junction is smaller than maximum interlocking force. Whether this situation occurs depends on the flexibility of the GGR. The main argument for not considering junction strength in conventional design is that the manufacturers produce their GGR in such a way that short-term strength of junctions is much higher than their *averaged* loading in the soil. However, in the light of our discussion, there would not be a sufficient factor of safety. Depending on degradation and flexibility, long-term strength of the junction might very well be a relevant material resistance concerning the behavior of the GGR in the long run.

5. THE CONCEPT OF JACOBS (2016)

5.1 The concept

The limit state equation 5 does not afford any deeper insight into the interplay between junction strength, flexibility of longitudinal ribs and soil strength. The questions are how to include the displacement dependent failure of junctions, which may occur at a loading level well below their tensile-shear strength and how to relate the interlocking force to the stiffness of the longitudinal ribs. Having answered these questions, one can estimate to which extend an approximate description of GGR behavior is valid, which excludes stiffness of ribs and failure modes of junctions. Jacobs (2016) and Jacobs et al. (2015, 2016) undertook a detailed study of the anchorage of product variants of a welded polymeric GGR to provide such an insight by a theoretical simulation based on small-scale pull-out tests and checked by large-scale pull-out laboratory tests and field measurements. A discrete-element model adapted to the various geometric conditions in an anchor trench (e.g. including deviation force at the top and toe) was used. Among the many attempts to simulate pull-out behavior (an overview is given in (Jacobs, 2016)) some have been of importance for the development of the concept (Wilson-Fahmy et al. 1993, Ziegler et al. 2004, Müller 2011, 2014).

The simulation is based on experimentally determined characteristic functions and cut-off parameters, which describe the interaction of the soil with transverse ribs, longitudinal ribs and junctions. The functions are: 1. Displacement versus interface friction force. 2. Displacement versus interlocking force. 3. Displacement versus interface friction force for the sliding trench failure, where the GGR together with the overlying soil layer slides on the underlying soil layer. A series of all three functions must be determined, because of the dependence of the functions on soil properties and normal stress. 4. Elongation versus tensile force for the longitudinal ribs.

The interlocking force, which the junction can sustain, is either limited by its tensile-shear strength or by a critical displacement. The effect of the limited strength of the junctions was included into the model by limiting the range and domain of the function No. 2. Hence, a cut-off parameter for range and domain of this function expresses failure of junctions. Going beyond a cut-off-value indicated to the numerical simulation that interlocking at the respective transverse rib no longer contributes to the interaction of soil and GGR. Similarly, long-term tensile strength of the longitudinal ribs was included by a cut-off parameter for the possible range of tensile forces described by function No. 4.

The functions and cut-off parameters were determined by many small-scale pull-out and shear tests for sand and gravel at normal stresses of 20, 50 and 100 kN/m². The simulation was checked by comparing its result with that of large-scale pull-out experiments. Since the model can not only be used to determine the ultimate limit state of pull-out but also displacements in the state of serviceability, the simulation was checked by in-field measurements of displacements of an installed GGR. For various geometries of anchor trenches as well as for various embedded lengths, normal stresses and tensile loads, the pull-out resistances were calculated in a comprehensive parameter study and compared with the determination of the pull-out resistances according to a conventional design method. The two types of failure modes of the anchorage (pull-out and sliding) were included by comparing the balance of the forces evaluated at each state of loading for each mode and switching accordingly from one mode to the other (details are given in (Jacobs 2016), section 5.2.1.1, p. 123). Deviation forces were included by a position dependent normal stress above and below the GGR (Jacobs 2016b).

5.2 Failure criteria for junctions

Peeling failure depends on displacement. Therefore, in addition to a permissible tensile-shear strength a permissible displacement of the junctions was introduced by Jacobs (2016). At which displacement and at which peeling strength the junctions fail by peeling was determined from various pull-out tests with only one transverse rib. The critical displacements indicated in Figure 3 were used for the simulation. A clear influence of the normal stress could not be found. However, for a given type of junction, the critical displacement increased with its initial tensile-shear strength.

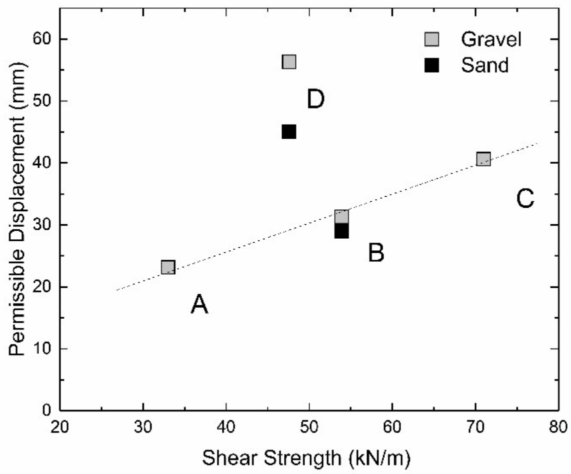

Figure 3. Displacement at which junctions failed in a small-scale pull-out test (sand 0/2 and gravel 0/32) as function of initial junction tensile-shear strength (from Jacobs (2016)). Variants A, B, C and D of the welded GGR were tested.

Such a critical displacement, which is short-term, has to be reduced in view of the long-term behavior of junction strength as discussed in section 4.2.2. For this, the relation between the critical displacement, which leads to peeling failure, and the initial tensile-shear strength of the junction, as indicated in Figure 3, may be used. If a long-term value for the tensile-shear strength is established (Eq. 6), an associated long-term cut-off parameter for the permissible displacement can be extrapolated from such a relation. The data in Figure 3 were obtained by performing pull-out tests on GGR-specimens with only one transverse rib (Jacobs 2016, Jacobs et al. 2015, 2016). Therefore, long-term values of the critical displacement may be derived by applying the same test and analysis of the test results on specimen with only one transverse rib, which, however, have been properly aged. Reliable cut-off values for long-term tensile-shear strength and long-term critical displacement must be implemented into the numerical model to get a full assessment of the repercussions of junction failure.

5.3 The stiffness of an anchorage

A linear relation between pull-out force R and the parameters tanφ, σ_n or L (Eq. 1) is only valid as an approximation over a small range of parametric values in case of flexible GGR. The interaction coefficient λ depends on the chosen range and it is, therefore, difficult to determine it experimentally. When λ is determined in small-scale pull-out (ASTM 6706) or pull-through tests (DIN 60009) with a small box, results for the pull-out resistance R are necessarily incorrect when Eq. 1 is applied to long embeddings. The question arises to what extent this is the case.

Laying aside the problem of determine λ and the specific effects of the geometry of a trench, the following conclusions can be drawn regarding the validity of a conventional description of the behavior of a GGR anchored into a layer of soil. If there is, for a given embedded length, a decrease of the stiffness of the ribs and the contribution of the interface friction force, then the reliability of a conventional design should decrease. For a given stiffness and contribution of the interface friction effect, the conventional approach will become the less reliable the more the imbedded length is increased. This is because the ratio of the length, which is really mobilized for a given soil material and normal stress, to the overall embedded length will become smaller with increasing overall length. Thus, the conventional approach is the more reliable the stiffer the longitudinal ribs, the greater the share of friction force on the pull-out resistance and the smaller the anchorage length. A conventional design is trivially correct, if the GGR is rigid and only interface friction forces occur (and the junctions are indestructible). Conversely, if the longitudinal ribs are highly extensible, the fraction of interlocking to friction forces is high and the anchorage length is large, the conventional approach might not yield safe results.

Stiffness of the longitudinal ribs was quantified by Jacobs (2016) by an “averaged long-term tensile stiffness” $J_{m,\infty}$ [kN/m], which was determined as follows. Tensile-force (per unit width of GGR) versus strain behavior of a GGR depends on temperature and deformation velocity. From creep testing tensile-force-strain-curves for various temperatures and velocities may be determined (isochronous curves). Taking the tensile-force-strain-curve for the temperature of application and the velocity related to the lifetime of the construction and using a linear approximation, the slope of the line gives the relevant averaged tensile stiffness. Dividing this quantity by the reduction factors for the tensile strength of the longitudinal ribs (see section 4.2.2) gives $J_{m,\infty}$. The share of the friction force is the ratio $\rho_{md} = \lambda_F / \lambda$ [dimensionless] as determined in small-scale pull-out tests with and without transversal ribs. Anchorage length is again designated L [m]. Jacobs (2016) introduced a parameter called “stiffness of the anchorage” k_R [kN/m²] as presented in the following equation:

$$k_R = (J_{m,\infty} \times \rho_{md}) / L \qquad [7]$$

According to the reasoning in the paragraphs above, it is expected, that in cases of an anchorage with small k_R a conventional approach does not correctly represent the behavior of the GGR. This can be quantified as follows: The anchorage resistance is the tensile force, which the anchored GGR can sustain. If for a given anchorage the ratio r of the anchorage resistance, as calculated by the numerical model, to the anchorage resistance, as calculated by a conventional approach, is ≥ 1, then the conventional approach is considered to be safe compared to the numerical simulation. If r < 1, then it is unsafe. Hence, if k_R is small, then r < 1 is expected.

Jacobs (2016) compared a conventional description of GGR behavior with his approach for many different cases of anchor trench geometries and loading conditions. Hence, the reliability of the conventional method was checked for the given GGR in a parameter study varying these conditions. Figure 6.22 in reference (Jacobs, 2016) shows the results of this study: for each of the many cases studied, r is plotted as function of the respective anchor trench stiffness k_R. In various cases the conventional approach was not safe, in many others it was safe. For the studied GGR with its specific stiffness and share of friction, it was found that in all cases the conventional approach was safe (r ≥ 1) for an anchorage stiffness greater than 1820 kN/m², i.e. r was greater than 1 independent of the variation of the geometry of the anchor trench, the slope angle and the normal stress. Whereas a conventionally calculated anchorage resistance had to be reduced by an additional reduction factor of 1.67 to avoid problems in cases where the anchorage stiffness was < 1820 kN/m², i.e. variation of geometry of the anchor trench, slope angle and normal stress gave in many such cases values of r smaller than 1 and in some cases even as small as 0.6. It follows that the flexibility of the longitudinal ribs may have a significant influence on the pull-out resistance of a GGR embedded into a layer of soil at least in cases where this anchorage has low stiffness. (Besides, sliding of the GGR was the relevant mode of anchor trench failure in almost all simulated cases. Sliding occurred after mobilization of friction and bearing resistance due to an intimate contact between soil and GGR.)

In his parameter study, Jacobs (2016) concentrated on the difference between the numerical model and conventional design due to the flexibility of the longitudinal ribs and the geometry of the anchor trench. Therefore, he used the high short-term values of the cut-off parameters of tensile-shear strength and permissible displacement of the junctions. Hence, junction failure had essentially no influence on the results. However, this influence becomes relevant when the short-term junction strength and permissible displacement is significantly reduced to due the degradation effects discussed in section 4.2.2.

5.4 Failure due to limited strength of junctions (Müller 2011, 2014)

Failure of degraded and overloaded junctions does not necessarily lead to a full-scale failure of the anchorage. The consequences of junction failure depend on the flexibility of the longitudinal ribs and the overall embedded length. This may be understood as follows. A tensile force leads to a certain configuration of displacements and forces along the mobilized part of the GGR. Let the GGR be subjected to a tensile force in such a way that the junctions in the front area of the embedding are loaded at the limit of their strength or permissible displacement. If the strength or permissible displacement of the junctions is reduced, the zipper mechanism will start. However, interface friction of the intact longitudinal ribs remains mobilized. Hence, the tensile force, which drives the zipper mechanism, is continuously reduced due to the friction of longitudinal ribs. If there is still a large enough mobilizable part of the GGR at the end of the embedding, one gets a configuration along this intact and mobilizable part, which is equivalent to an original configuration, however, at a lower tensile force. If this reduction in tensile force fits to the reduced strength or permissible displacement of the junctions, the zipper will stop. If not, the zipper will run through and pull-out of the entire GGR will occur. The basic question regarding a very long service life that should be asked, is: what are the values for long-term strength and long-term permissible displacement of the junctions of the GGR by which the zipper condition can be prevented? This can only be decided on the basis of the experimental study of long-term junction strength, as describe in section 4.2.2 and 5.2 and of the simulation of GGR performance as describe in section 5.1.

6. CONCLUSIONS

The strength of GGR junctions is limited and degrades over the long term. A safe design must consider this effect. The tensile-shear-strength of a junction can be determined using method a or method b of ASTM D 7737. Small-scale pull-out tests with a GGR-specimen with only one transverse rib can be used to check whether other failure modes are relevant. Peeling strength can be indirectly characterized by such pull-out tests in terms of critical displacement of transverse ribs. GGR junctions will degrade at least as much as GGR ribs in the long run. These tests can be applied to properly aged specimens to determine the long-term tensile-shear strength and/or the long-term critical displacement. In special cases, it might be necessary to use long-term tensile shear testing or long-term small-scale pull-out testing to assess the effect. These long-term values can be used as input to numerical discrete element models to simulate GGR-soil interaction. Such numerical models must be used because they include the effect of the flexibility of the longitudinal ribs. It is the flexibility of the longitudinal rib, which determines the distribution and the amount of loading of the junctions, hence, the length of the truly mobilized part of the GGR. Using such models, it can be determined to which extend a conventional description of GGR behavior is safe and to which extend degradation of junctions can lead to failure of the anchorage.

REFERENCES

ASTM D 7737. Standard Test Method for Individual Geogrid Junction Strength, *American Society for Testing and Materials*, West Conshohocken, Pennsylvania, USA.

ASTM 6706. Standard Test Method for Measuring Geosynthetic Pullout Resistance in Soil, *American Society for Testing and Materials*, West Conshohocken, Pennsylvania, USA.

ISO/TR 20432. Guidelines for the determination of the long-term strength of geosynthetics for soil reinforcement, *International Organization for Standardization (ISO)*, Geneva, Switzerland.

DIN 60009. Geokunststoffe - Prüfung und Bestimmung des Verbundbeiwerts mit Boden im Herausziehversuch, *Deutsches Institut für Normung (DIN)*, Berlin, Germany.

BAM (2013). Zulassung für Bewehrungsgitter aus Kunststoff für Deponieoberflächenabdichtungen, *Amts- und Mitteilungsblatt* 43:16-38. https://opus4.kobv.de/opus4-bam/frontdoor/index/index/docId/8. Accessed 30 August 2019.

Bathurst, R., Ezzein, F. (2016). Geogrid pullout load–strain behaviour and modelling using a transparent granular soil, *Geosynthetics International* 23:271-286. https://doi.org/10.1680/jgein.15.00051.

Bräu, G., Herold, C. (2011). *Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements – EBGEO*, Ernst & Sohn, Berlin, Germany.

Dyer, M.R. (1985). Observation of the stress distribution in crushed glass with application to soil reinforcement, Dissertation, University of Oxford, Great Britain.

Ezzein, F. M., Bathurst, R. J. (2014). A new approach to evaluate soil-geosynthetic interaction using a novel pullout test apparatus and transparent granular soil, *Geotextiles and Geomembranes* 42:246-255. https://doi.org/10.1016/j.geotexmem.2014.04.003.

Ezzein, F. M., Bathurst, R. J., Kongkitkul, W. (2015). Nonlinear load–strain modeling of polypropylene geogrids during constant rate of strain loading, *Polymer Engineering and Science* 55:1617-1627. https://doi.org/10.1002/pen.23999.

Jacobs, F. (2016) Interaktionsmodell zur Bemessung von Verankerungsgräben mit Geogittern, Dissertation, RWTH Aachen University, Aachen, Germany. https://d-nb.info/1130327353/34. Accessed 11 September 2019.

Jacobs, F., Ziegler, M., Vollmert, L., Ehrenberg, H. (2014) Explicit design of geogrids with a nonlinear interface model, *Proceedings of the 10th International Conference on Geosynthetics*, German Geotechnical Society (DGGT), Essen.

Jacobs, F., Ziegler, M., Vollmert, L., Ehrenberg, H. (2016) Design of geogrid pullout with a nonlinear interaction model, *GeoAmericas 2016 - 3rd Pan-American Conference on Geosynthetics*, Minerva-Technology, Resources, and Information, Jupiter, Florida.

Jacobs, F. (2016b). Interaction model for design of geogrid pullout, *Proceedings of 6th European Geosynthetics Congress*, Ljubljana, Slovenia, 794-803.

Jewell, R. A. (1996). *Soil reinforcement with geotextiles*, Thomas Telford, London.

Koerner, R. M. (2012). *Designing with Geosynthetics*, 6th ed. Xlibris Corporation, http://bookstore.xlibris.com/, Accessed 11 September 2019.

Kupec, J., McGown, A., Ruiken, A. (2004). Index testing of the junction strength of geogrids, *Proceedings of the Third Asian Regional Conference on Geosynthetics, Now and Future of Geosynthetics in Civil Engineering*, Seoul, South Corea, 797-802.

Kupec, J., McGown, A., Ruiken, A. (2004b). Junction strength testing for geogrids, *Proceedings of the Third European Geosynthetics Conference, Geotechnical Engineering with Geosynthetics*, Deutsche Gesellschaft für Geotechnik (DGGT) und Technische Universität München (TUM-ZG), München, 717-722.

Lopes, M. L. (2002) Soil-geosynthetic interaction, in: Shukla, S. K. (ed), *Geosynthetics and their applications*, Thomas Telford, London.

Müller, W. W. (2011). Zur Bemessung der Verankerung von Bewehrungsgittern aus Kunststoff beim Schutz von Böschungen vor hangparallelem Gleiten, *Bautechnik* 88:347-361. https://doi.org/10.1002/bate.201101472.

Müller, W. W. (2014). Long-term pull-out resistance and materials properties of geogrids, *Proceedings of the 10th International Conference on Geosynthetics*. German Geotechnical Society (DGGT), Essen.

Palmeira, E. M. (2004). Bearing force mobilisation in pull-out tests on geogrids, *Geotextiles and Geomembranes* 22:481-509.

Palmeira, E. M. (2009). Soil-geosynthetic interaction: Modelling and analysis. *Geotextiles and Geomembranes* 27:368-390.

Prandtl, L. (1920). Über die Härte plastischer Körper. *Nachrichten von der Gesellschaft der Wissenschaften zu Göttingen, Mathematisch-Physikalische Klasse*. Göttingen, Germany, 74-85. https://eudml.org/doc/59075. Accessed 19 September 2019

Prandtl, L. (1921.) Über die Eindringungsfestigkeit (Härte) plastischer Baustoffe und die Festigkeit von Schneiden. *Journal of Applied Mathematics and Mechanics/Zeitschrift für angewandte Mathematik und Mechanik* 1:15-20.

Reissner, H. (1924). Zum Erddruckproblem, *Proceedings of the First International Congress of Applied Mechanics*, 295-311.

Sieira, A. C., Gerscovich, D., Sayão, A. (2009). Displacement and load transfer mechanisms of geogrids under pullout condition, *Geotextiles and Geomembranes* 27:241-253. https://doi.org/10.1016/j.geotexmem.2008.11.012.

Swan Jr., R. H., Yuan, Z. A. (2016). Theoretical Analysis of the Maximum Load Transferred to the Junctions of a Geogrid Confined in Granular Soil, *Proceedings of the Geo-Chicago 2016: Forging a Path to Bona Fide Engineering Materials*, ASCE Library, 37-48.

Wilson-Fahmy, R. F., Koerner, R. M. (1993). Finite element modelling of soil-geogrid interaction with application to the behavior of geogrids in a pullout loading condition, *Geotextiles and Geomembranes* 12:479-501. https://doi.org/10.1016/0266-1144(93)90023-H.

Wilson-Fahmy, R. F., Koerner, R. M., Sansone, L. J. (1994). Experimental Behavior of Polymeric Geogrids in Pullout, *Journal of Geotechnical Engineering* 120:661-667. https://doi.org/10.1061/(ASCE)0733-9410(1994)120:4(661).

Ziegler, M., Timmers, V. (2004). A new approach to design geogrid reinforcement, *Proceedings of the Third European Geosynthetics Conference, Geotechnical Engineering with Geosynthetics*, Deutsche Gesellschaft für Geotechnik (DGGT) und Technische Universität München (TUM-ZG), München, 661-667.