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Reliability design applied to geosynthetics reinforced retaining walls

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ABSTRACT: Geosynthetic reinforced soil walls (GRSW) have been widely used as retaining structures. In the traditional design of geosynthetic reinforced soil walls, Allowable Stress Design (ASD) is used to address the uncertainties. However, it cannot explicitly consider the uncertainties in a systematic way in the design process; especially geotechnical uncertainties which are typically project specific. The present paper an example to demonstrate the significance of reliability-based design method for GRSW structures.

1 INTRODUCTION

Geosynthetic reinforced soil walls (GRSW) are implemented in geotechnical engineering projects all around the world thanks to the redundancy of stability mechanisms and their ductile performance against various loading and foundation deformation.

According to Zannoni (2016), while GRSW are widely used in geotechnical engineering, the performance prediction from a design model can be highly uncertain because of the difficulties in accurately determining geotechnical and loading parameters in the design. Failure to consider such uncertainties could lead to either expensive over-design or under-design, which may prolong or reduce the construction period and in the worst-case scenario may fail to meet the performance requirements. Usually loading conditions are known quite well to a certain extent, hence the challenge is usually to define the geotechnical properties of the soils involved, either in situ soils (typically the soil at the back the GRSW body and the foundation soil) or imported soils (typically the GRSW fill).

The design of GRSW is based on the properties of the geosynthetic reinforcement and on the geotechnical properties of the soils, since the structure is made up of engineered compacted soil and geosynthetics.

Depending on the size and importance of the project, and the geology of the site, the geotechnical investigation might vary in methodology and number of in-situ and laboratory tests to determine, to a certain degree of confidence, the soil properties.

Working stress and now partial factor design-based codes have the challenging job to try ensuring the safety of the design within a certain degree of safety, based usually on the type of structure.

What is usually very difficult and requires experience and knowledge is setting the correct values of the geotechnical parameters of soils: what is the right value of friction angle or cohesion for the actual soil strata? Even if the design is formally correct, when these values are incorrect, the whole design can be compromised.

The aim of the present paper is to demonstrate that high confidence in the design of a GRSW can be obtained even with high variabilities of soil parameters, consequent to poor/inexpensive geotechnical investigation.

2 INTERPRETATION OF GEOTECHNICAL INVESTIGATION

According to Zannoni (2016), in the traditional design of GRSW, Allowable Stress Design (ASD) is used to address uncertainties in the soil parameters, adopting an experienced calibrated Factor of Safety (FoS) in the design process. However, the deterministic methods rely significantly on engineering judgment and does not explicitly consider uncertainties, particularly the geotechnical uncertainties which are project and site specific. ASD methods can results in over-conservativeness, inconsistence, and empiricism in the design process. In recent years, reliability-based methods have demonstrated to be an effective approach for design of geosynthetic reinforced soil structures under various failure modes and loading conditions using a probabilistic approach (Sayed et al. 2008, Yang et al. 2010, Miyata & Bathurst 2012, Basha & Babu 2014, Chen et al. 2016).

For geotechnical problems which do not depend on extreme micro-scale soil structure, i.e. which involve some local averaging, it can be argued that the behaviour of the spatially random soil can be closely represented by a spatially uniform soil which is assigned the 'effective' properties of the spatially random soil. These effective properties representation have been successful in the past, for a variety of geotechnical problems, by determining the effective uniform soil as some sort of average of the random soil. If the above arguments hold, then it implies that the spatially random soil can be well modelled based on uniform soil properties. The problem becomes to define the appropriate effective soil properties.

In practice, the values of φ and γ are obtained through site investigation and laboratory tests.

If the investigation is thorough enough to allow spatial variability to be characterised, an effective soil property can, in principle, be determined using random theory combined with simulation results. However, the level of site investigation required for such a characterisation can be very expensive. Generally, the geotechnical engineer may base the design on a single estimate of the friction angle and unit weight. In this case, the accuracy of the prediction depends very much on how well the single estimate approximates the effective value.

Sometimes, due to site restriction, budget or quality of the results, geotechnical investigation provides incorrect or more precisely not reliable information (Jacobs et al. 2009). Indeed, geotechnical investigation is characterised by uncertainties which can be related to the type of soil, quality of the investigations, number of values, lack of knowledge and gross errors.

All of these uncertainties influence the interpretation of the soil properties and they could lead to the incorrect assumptions which are the input for the design. An incorrect geotechnical investigation will surely result in an incorrect geotechnical design.

In order to ensure a certain degree of confidence in geotechnical investigations, it is usually recommended to get correlated testing which can be used to check if the results are correct. Often field test which represent the bulk of the geotechnical investigations are checked against laboratory tests (which are performed on samples obtained in very specific points), in order to calibrate the bulk of the information.

2.1 *Variability of geotechnical parameters and characteristic value*

It is known that any geotechnical engineer evaluating a geotechnical investigation will determine different design values based on their knowledge, experience with the testing method, the site, the geology and the operators (Bond et al. 2008). In EN 1990 (2002) the engineer is required to define the characteristic value (X_c) of materials, including soil: when a low value of material or product property is unfavourable, X_c should be defined as the 5 % fractile value; when a high value of material or product property is unfavorable, X_c should be defined as the 95 % fractile value. This definition works well with man-made materials such as concrete or geosynthetics, however for soil this might not apply since some parameters might have a coefficient of variation up to 60 % (Phoon 1995), as reported in Table 1. For this reason, in EN 1997-2 (2007), dealing with geotechnical designs, the characteristic value is defined as "cautious estimate", which is open to any interpretation.

Table 1. Coefficient of variation (COV) of geotechnical and man-made-materials (Bond et al 2008)

Material	Parameter	COV
Soil	coefficient of shearing resistance	5-15%
	effective cohesion	30-50%
	Undrained strength	20-40%
	Coefficient of compression	20-70%
	Weight density	1-10%
Concrete	Resistance of beams and columns	8-21%
Steel		11-15%
Aluminium		8-14%

A further complication is the volume of soil involved in the design. If it is agreed that soil is a variable material in its performance, indeed it will vary spatially (its variation in time is not considered in this paper). To calculate the pile tip resistance a small amount of soil is considered, and it is well defined as the depth is known; whereas for a GRSW stability the soil that should be considered includes the wall fill, the back soil and the foundation soil, which might means thousands m3 of soil strata with different history and behaviour.

The Eurocode 7 (EN 1997-2 2007) amends the definition of characteristic value to "a cautious estimate of the value affecting the occurrence of the limit state", leaving it the choice to the designer based on the category of the structure.

The use of reliability design is highly complicated since, in order to consider the use of a probabilistic approach, the designer should have repeated correlated values. In a simple example, the amount of data required for a slope stability analysis considering only one uniform and homogenous soil (which is already a simplification) will require to get more than 5 samples of friction angle, cohesion and soil density for which the variance is known. In some instances, where the magnitude of the project allows a thorough geotechnical investigation (i.e. power stations, airports, industrial areas) the amount of data might allow the designer to consider a probabilistic design approach.

2.2 *Variability of geosynthetics for soil reinforcement*

Geosynthetics materials are man-made materials manufactured under strict quality control procedures in order to ensure the highest confidence level of the required properties.

According to SATR 20432, reduction Factors RF are required for tensile creep and chemical degradation; both these RF are based on extrapolation methods which takes into account the uncertainties in extrapolation over long durations: uncertainties of creep testing and uncertainties of accelerated chemical tests.

The ability to perform repeated testing in a controlled environment has enabled geosynthetics to be characterised by a coefficient of variation less than 10 % at 120 years design life.

A commercial bonded polyester geogrid (made up of a regular array of composite geosynthetic straps, nominally interconnected laterally to form a geogrid with high unidirectional strength) is characterised by the properties reported in Table 2 (from Linear Composites 2015). From the results it is clear the high reliability of geosynthetic properties, both in short term (low CV value of the tensile strength) as well as in the long term (RF is just 1.05, well below the normal range value of 1.2 - 1.3).

Table 2. Short term characteristics of geogrid.

		Characteristics	Value
Declaration of Performance (DOP)	Tensile Strength	Mean	206
		95% Confidence	201.1
Based on 50 tests		Mean	208
		St dev.	2.43
		95% Confidence	204
		CV	1.17%
	Factor of Safety	f_s	1.05

2.3 *Cost of laboratory and in-situ tests*

Experts in the field of geotechnical engineering are much aware of the research compiled by Mott MacDonald and Soil Mechanics Ltd in 1994 (Fig. 1), reviewing past projects, projecting the cost of the site investigations over the increase in costs and increase in construction cost. If the cost of the site investigation is less than 50 % of the total construction tender costs, according to the research, the increase in actual construction cost can increase up to 50 % and even to 100 % in some cases.

However, the owner or the project manager has a very limited budget at this early stage of the project as every expenses will be recovered only once the project is completed, which could be in several years; keeping also in mind that there is always the possibility that the site is not suitable for the project and relocation, change in scope or lastly, cancelling of the project could happen, resulting in a complete loss in investments.

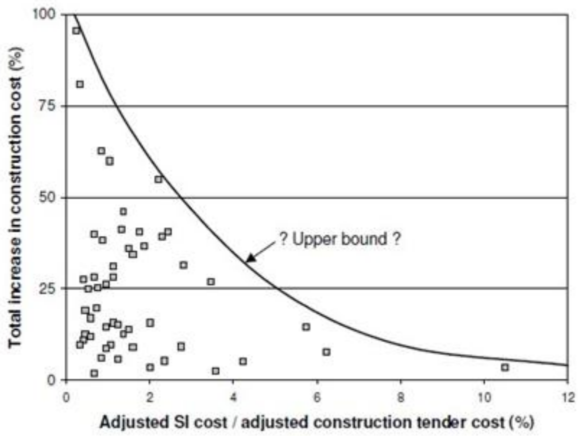

Figure 1. Impact of site investigation (SI) expenditure on UK highways contracts (Mott MacDonald and Soil Mechanics Ltd 1994)

The major issue is the foundation as for retaining structure knowledge up to twice the width is required (SAICE 2010). Usually site investigation is concentrated by critical structures such as bridges or highly critical structures such as tailings dam walls. GRSW in civils and mining are used to contain fill materials as usually the toe exceeds the road reserve or interfere with other structures. This is often finalised in preliminary design or even in final design, although the geotechnical investigation might have been done already in pre-feasibility to have a general understanding of the site. Therefore, it could happen that no information is available within the area of the wall and interpretation of data is required applying a certain degree of caution.

Considering a GRSW 100 m long, 8 m high, it will mean that the footprint area of interest is about 5000 m^2. The project could be overlaying a uniform area; therefore, one borehole will be enough, or the area might be characterised by the presence of pockets of very soft soil, requiring special testing. Nevertheless, generally for 800 m^2 facia wall the cost to build varies between R 1000 / m^2 to R 3000 / m^2. Considering an average cost, the structure is in the order of R 1.6 million. Using Figure 1, fixing a 1 % in site investigation, it will be feasible to invest R 16,000. Considering the structure and author's experience the cost for a thorough investigation is rated at about R 300,000 which compared to the budgeted amount is more than 20 times higher. Therefore, the designer will face a design without having enough knowledge of the foundation which will push the design values on the cautious side, therefore assuming worst material properties which surely will increase the base of the structure due to the poor bearing capacity.

Furthermore, the costing of the backfill in terms of the example it will be of about R 40,000. While grading, density and compaction are standard test available on site with high reliable value, direct shear testing and triaxial testing is often a laboratory testing which requires experience personnel, otherwise the results might be compromised and furthermore, such testing are not performed on site but only by specialised lab, with high cost and most important, waiting time which might push the contractor to proceed with construction at risk.

3 RELIABILITY APPROACH TO GRSW DESIGN

According to Zannoni (2016), the current design philosophy for GRSW is based on limit state analyses applying partial factors. This design approach enables the study of the behaviour of the structure at different state (usually ultimate limit states and serviceability limit states) and it allows to apply reduction factors to the single parameters rather than one factor of safety for all (working stress approach).

The scope of any design philosophy is to ensure that the design is safe, but how safe is it? This is a hidden issue as a temporary structure should not be characterised by the same margin of safety that a bridge abutment requires. In terms of reliability approach, reliability is linked to probability, therefore a reliability index of 3 generally means a probability of failure of 1 x 10^{-3} (Fig. 2).

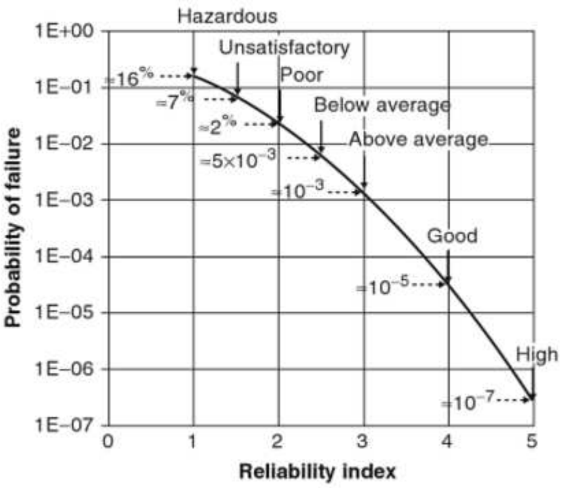

Figure 2. Probability of failure vs reliability index (from US Army Corps of Engineers 1997)

Bathurst (2018) argues that the internal stability design for MSE walls in the UK is based on a partial factor approach in which factors are applied to soil and reinforcement material properties and to load contributions in different combinations to ensure safe designs. Geotechnical foundation design codes in North America adopt a load and resistance factor design (LRFD) approach which has been used by structural engineers for decades in Canada and the USA.

The intent of a properly calibrated limit state design equation expressed in a LRFD framework is to ensure that a target maximum probability of failure will not be exceeded. However, the load and resistance factors that appear in LRFD codes have been selected largely by fitting to factors of safety used in allowable (working) stress design (ASD) past practice. Whether a designer uses a partial factor approach as in the UK or a LRFD approach as in North America, the margin of safety expressed probabilistically is unknown. This leads to the conundrum of a limit state being satisfactory when viewed from a factor of safety point of view but unsatisfactory from a probability of failure perspective. In conventional allowable stress design, the ratio of nominal resistance to nominal load defines the factor of safety; the resistance term is adjusted so that the factor of safety satisfies a minimum acceptable value. A parameter to quantify margins of safety in geotechnical engineering is the reliability index β. The relationship between probability of failure and reliability index is:

$$P_f = 1 - F(\beta) \quad (1)$$

Bathurst (2018) argues that values of β = 2.33 and 3.09 correspond to probabilities of failure of 1 / 100 and 1 / 1000, respectively. The smaller β value is recommended as the target minimum reliability index for internal limits states design and LRFD calibration for GRSW. This value may appear small but GRSW walls are highly strength-redundant systems. In other words, if one reinforcement layer fails, other layers can compensate and thus system failure is unlikely. If a reinforcement layer is designed to just satisfy a target reliability of β = 2.33, the corresponding factor of safety can be as high as 1.70.

4 EXAMPLE OF RELIABILITY ANALYSIS OF GRSW

Acknowledging the use of geosynthetics, characterised by a low coefficient of variation compared to geotechnical parameters, will lead to a safer and efficient design. In order to illustrate the influence of the geosynthetics in a soil reinforcement system, the 10 m high wall is considered, with the input data in Table 3, which shows also the assumed variabilities of parameters. For sake of simplicity, only cohesionless soils are considered in this example. The reinforcement is assumed to be bonded polyester geogrids, for which typical values of RFs, f_{ds}, and $f_{po,}$ are assumed (BBA 2010). The vertical spacing between the reinforcements is set as 0.6 m. Surcharge is a uniform load applied along the top horizontal surface of the wall with a set value of 20 kPa.

The first analysis aimed to calculate the tensile strength in the reinforcement which depends on the properties of the fill. In this example, the cohesion was omitted to reduce the calculation as it is good practice to consider the backfill cohesionless.

Considering the tensile strength, the friction angle and the density stochastic variable with characteristic reported in Table 3.

Table 3. Input data for example calculation

Variable	Mean	CV	Distribution
T	90 KN/m	1%	Normal
φ_s	32°	10-40%	Log-Normal
γ_s	18 KN/m^3	5%	Normal

The percent variability for the ultimate tensile strength T_{ult} and the Reduction Factors of the reinforcement are assumed as 1 %, considering the chemical and biological degradation, installation damage and creep effects. Geometrical and load parameters are assumed with no variability.
The probability of failure varies from 0.17 % to 21.68 % (Fig 3). In terms of reliability index, the design moved from above average which has a reliability index of 2.94, very close to the SANS 10160-5 to an index of 0.79 which is a very hazardous design.

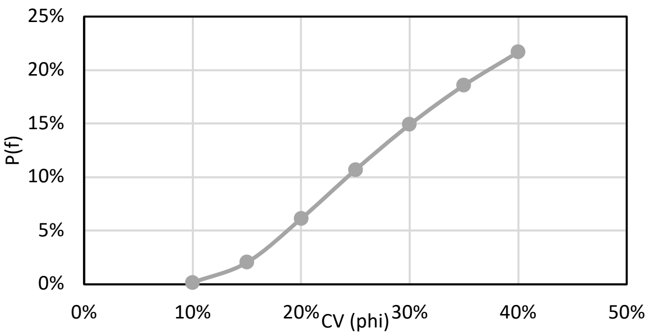

Figure 3. Variation of P(f) with the CV (φ_b)

In most cases the variability of the geotechnical properties is not known, while it is known the tensile strength of the geosynthetics. A further analysis varied the tensile strength of the geosynthetics in order to keep the probability of failure at less than 0.5 % or below average. From the results in Figure 4, the strength increases with the increase of CV from 90 KN/m to 200 KN/m. It should be mentioned that a CV of 40 % for the friction angle has brought the friction angle to almost 0 as the method will look for the worst possible combination of value. Such a low friction angle is only used to show the pattern.

While the tensile strength only affects the stability in terms of rupture, the length of the geosynthetics influence all the other factors of safety. The same approach is followed considering the variation of the foundation properties and the effect on the length of the geosynthetics.

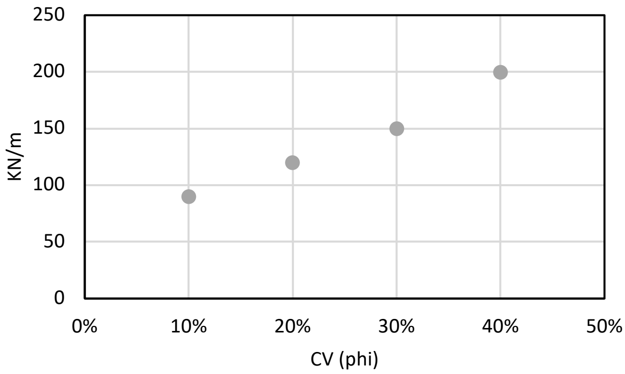

Figure 4. Variation of tensile strength with the CV (φ_b)

Table 4. Input data for example calculation

Variable	Mean	CV	Distribution
L	7m	1%	Normal
φ_f	28°	10-40%	Log-Normal

During analysis it appeared that the critical stability is the direct sliding along the base, therefore the following paragraphs will omit the other analysis for sake of synthesis.

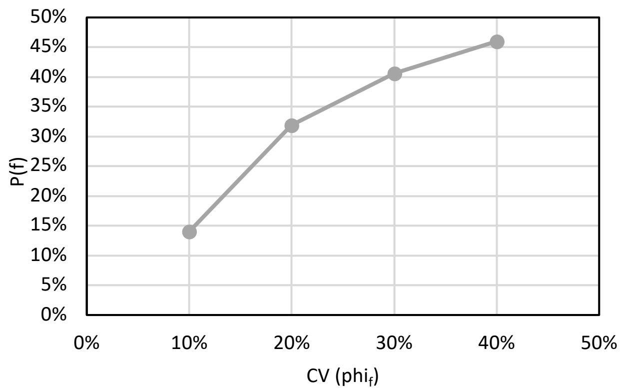

Figure 5. Variation of P(f) with the CV (φ_f)

Following the same pattern of the tensile strength, an increase in the variability for the friction angle at the base, increases the probability of failure from 15 % to more than 45 % (Fig. 5). In order to maintain a probability of failure within acceptable limits (< 0.5 %) at the increase of the CV (φ_f) the base will have to increase as shown in Figure 6.

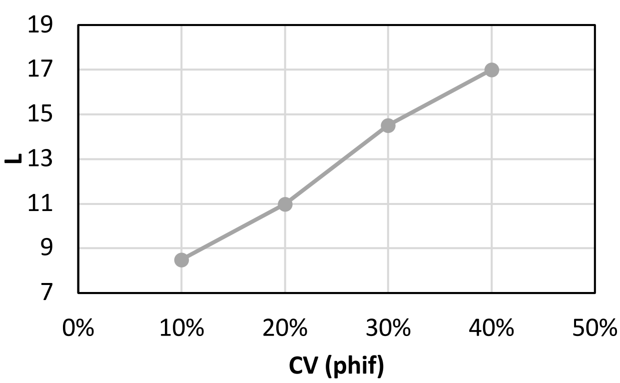

Figure 6. Variation of length strength with the CV (φ_f)

Based on the above simulation, it is evident that the inclusion of geosynthetics with very low coefficient of variability reduces drastically the probability of failure (Fig.7) from 0.09 % for a CV (T) of 1 % to a p(f) di 6.88 % for a CV (T) of 20 %.

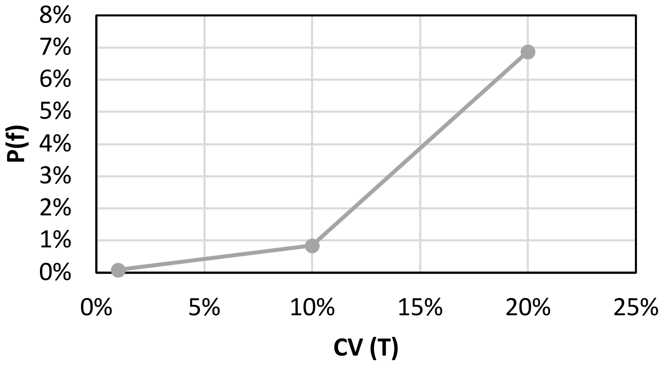

Figure 7. Variation of p(f) with the CV (T)

The aim of the design is to manage the risk of failure, which translates in minimising within acceptable costs the probability of failure.

5 CONCLUSIONS

To reduce the probability of failure in the above example the geotechnical investigation and the construction quality control would require about R 350,000 to R 500,000. Reducing the cost of the investigation and allowing the geosynthetic reinforcement to counteract the increase in the coefficient of variability to 20 %, the geosynthetics should be increase by 57 % in length and 20 % in strength which will generate an increase in costs of R 300,000.

In this extreme example a 20 % COV of the friction angle means a friction angle dropping from 32° to less than 25° which is a very remote possibility, the principle of increasing the geosynthetic performance (tensile strength) and quantity (length) produce a cost-saving to the cost of the project. Furthermore, especially in remote areas where geotechnical testing could require up to months to be performed, the cost saving of avoiding to hold the construction is priceless.

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