The 2016-update of the Design Guideline Basal Reinforced Piled Embankments

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The first piled embankment reinforced with geosynthetic reinforcement (GR) was constructed in 1972 in the in de Göta älv valley in Sweden ([8]). The first British piled embankments were constructed in the early 1980s, for example the foundation of an abutment of the Second Severn Crossing [10]. The Monnickendam Bus Lane is the first Dutch piled embankment, dating back to 2000. Since those days many basal reinforced piled embankments have been constructed worldwide.

A basal reinforced piled embankment (Fig. 1) consists of a reinforced embankment on a pile foundation. The reinforcement consists of one or more horizontal layers of geosynthetic reinforcement (GR) installed at the base of the embankment. The embankment is filled with for example crushed demolition waste aggregate or sand.

A basal reinforced piled embankment can be used for the construction of roads or railways when a traditional construction method would require too much construction time, affect vulnerable objects nearby or give too much residual settlement, making frequent maintenance necessary. Some piled embankments are long, like the Dutch 14 km long regional road N210 between Krimpen and Bergambacht, or the Dutch 3.5 km long bypass road near Reeuwijk [19]. Other piled embankments are short and for example constructed for transition zones between traditional embankments on soft soils and a founded structure or as an abutments for a viaduct.

The first edition of the Dutch design guideline for basal reinforced piled embankments CUR226 was published in 2010 [13] and adopted major parts from the German EBGEO (2010, [3]). Since those days, the knowledge about basal reinforced piled embankments has developed largely; a more reliable design method for the GR became available [17, 18], an adaptation to the Eurocode was needed and questions about pile cap design arose, making an update of CUR226 [1] necessary. This paper describes the highlights of the 2016-update of CUR226 [2].

GR DESIGN

The GR strain needs to be calculated to design the GR. Multiplying this GR strain by the GR stiffness gives the tensile force, which needs to be smaller than the long-term GR tensile strength. Most calculation models calculate the GR strain in two steps (Fig. 2a and b). Step 1 divides the vertical load into two load parts. One part (load part *A*) is transferred to the piles directly. This part is relatively large because a load tends to be transferred to the stiffer parts of a construction. This mechanism is known as ‘arching’. The second, residual load part (*B* + *C*) rests on the GR (*B*) and the underlying subsoil (*C*), see Fig. 2c.

Calculation step 2 determines the GR strain. Only the GR strips between each pair of adjacent piles are considered: they are loaded by *B*+*C* and may or may not be supported by the subsoil. The GR strain can be calculated if the distribution of load part *B*+*C* on the GR strip, the amount of subsoil support and the GR stiffness are known.

CUR226 (2010, [1]) used the calculation model of [21] for GR design. The German EBGEO had already adopted that model before. The Dutch made the same choice for Zaeske’s model because it matched some field measurements reasonably well. However, many more measurements became available since 2010. Van Eekelen et al. [18] showed that Zaeske’s model gives on average 2.5 times the strain measured in seven field cases and four laboratory series of experiments (Fig. 3a).

CUR226 (2016, [2]) uses the Concentric Arches model of Van Eekelen and Van Eekelen et al. [16, 17, 18]. This model was developed on the basis of a series of laboratory tests [14, 15]. Calculation step 1 consists of a set of 3D and 2D concentric arches as shown in Fig. 2a. The load is transported along the concentric arches. Smaller arches exert less load on their subsurface, large arches exert more load on their subsurface. The result is that a relatively large load is exerted on the pile caps (*A*) and the GR strips between adjacent piles, which matches measurements quite well. Figure 2b shows the load distribution on the GR strips between adjacent piles for step 2 as adopted in CUR226 (2016, [2]); when there is no subsoil support, or almost no subsoil support, the inverse triangular load distribution is used. When there is significant subsoil support, a uniform load distribution is used.

Fig. 3b shows that the GR strain calculated with the new model is on average 1.1 times the measured GR strain with a lower coefficient of determination, R^2, than shown in Figure 3a. The calculated GR strain is therefore almost a perfect match with the measured GR strain. CUR226 (2016, [2]) has therefore adopted the Concentric Arches model.

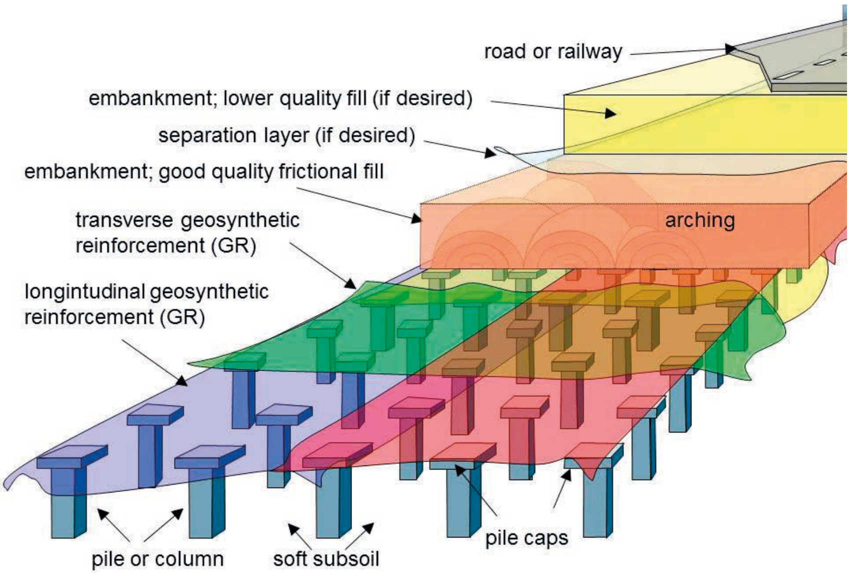

Fig. 1. A basal reinforced piled embankment (taken from [20])

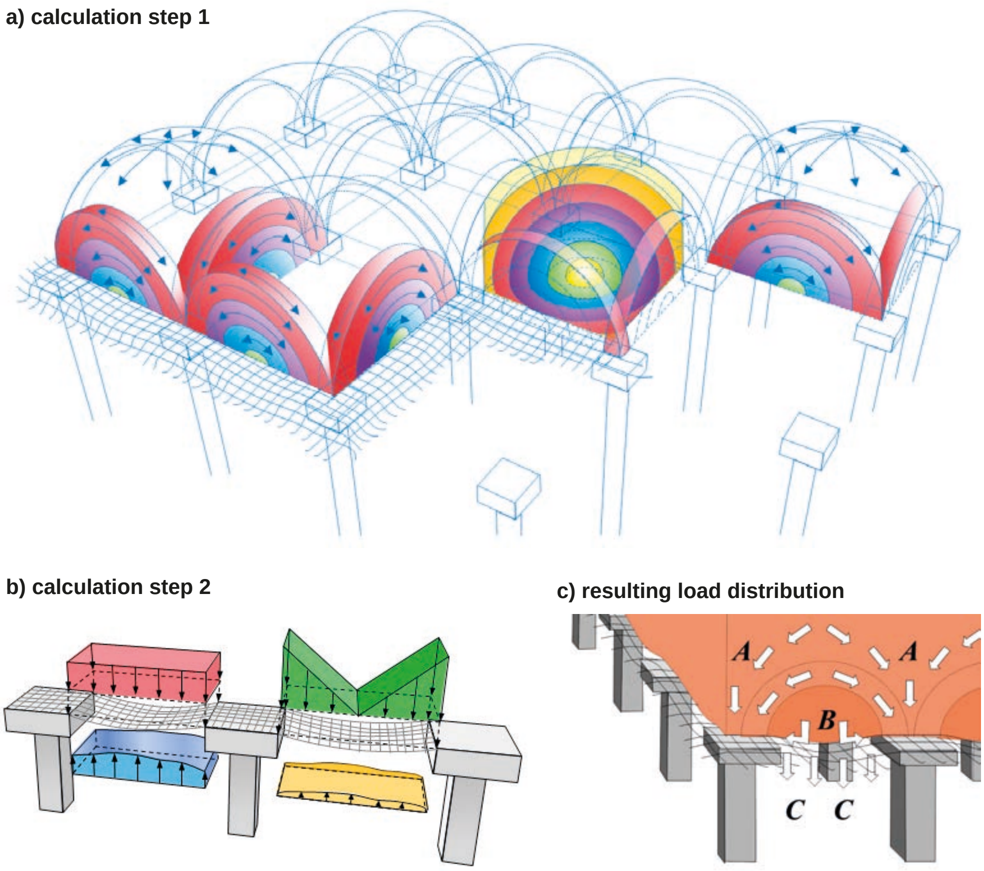

Fig. 2. The new Concentric Arches model [15, 16, 17, 18] consists of two steps: (a) step 1 calculates the load distribution in A and B + C and (b) step 2 calculates the GR strain that occures in the GR strip between adjacent pile caps (c) resulting load distribution (A, B, C) ((a) taken from [16], (b) taken from [18], (c) taken from [14])

MODEL FACTOR, LOAD AND MATERIAL PARTIAL FACTORS

One can debate whether a design guideline should adopt a model that nearly always gives a design on the safe side, as with the Zaeske model (Fig. 3a), or whether a design guideline should adopt a model that describes reality as well as possible (Fig. 3b) and consider safety separately. The Dutch CUR226 committee decided to adopt the new Concentric Arches model and to combine this with the inclusion of a model factor to cope with the uncertainty in the model. The value of the model factor was determined using the data points given in (Fig. 3b).

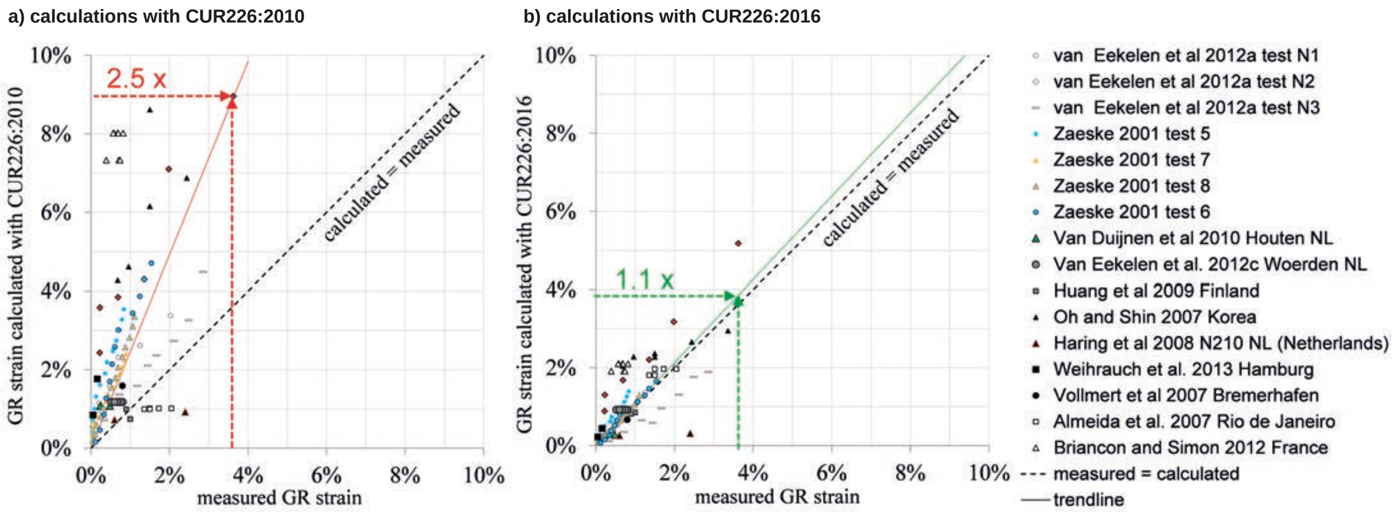

Fig. 3. Comparison calculations and measurements in seven field projects and four series of experiments. Van Eekelen et al., 2015 gives the sources of the references given in this picture, which are not given in the references of this paper due to space limitations. Calculations without safety factors.

Van Duijnen et al. ([11]) reported the safety analysis used to determine the model factor and the associated load- and material factors. Following the suggestions made in [4], they conducted a statistical assessment of the differences between the measured and calculated GR strains and then carried out Monte Carlo (MC) simulations for the SLS situation, for several reference cases in order to obtain the model factor. Multiplying this model factor by the GR strain calculated with characteristic values gives a value that is higher than the real GR strain in 95% of the cases. In other words, if the model factor is used, reality is worse than the calculation in 5% of the cases.

Subsequently, Van Duijnen et al. ([11]) determined three sets of partial material and load factors associated with the model factor for a level 1 design approach (the method with partial factors). They showed that using these factor sets satisfy the reliability indices β required for the three reliability classes of [4]. The resulting model and partial factors were adopted in [2]and are shown in Table 1.

Extensive calculation examples of GR design were included in the 2016-update of CUR226 [2].

LIMITATIONS

CUR226 (2016, [2]) was written for piled embankments with a basal geosynthetic reinforcement. The validation of the GR design rules was conducted with measurements in piled embankments with:

– a centre-to-centre (ctc)-pile spacing< 2.50 m;
– geogrids, in some cases combined with woven geotextiles (geogrid on top of geotextile);
– a groundwater level below or only just above the pile caps;
– $0.5 < H/(s_d - d_{eq}) < 4.0$; with H (m) the height of the embankment, s_d (m) the diagonal ctc-pile spacing and d_{eq} (m) is the diameter or equivalent diameter of the pile cap;
– vertical stresses on top of the GR above the pile caps up to 1450 kPa. In practice, however, some embankments of this type have already been realised with vertical stresses on the pile cap of 2000 kPa.

Table 1. Model factor and partial safety factors used for the design of the GR design in [2]
The calculated strain should be multiplied with the model factor γ_M, γ_f is a load factor, $F_d = \gamma_f \cdot F_k$, γ_m is a material factor, $X_d = X_k / \gamma_m$, a unit weight increase is not beneficial, hence the value of $\gamma_{m;\gamma}$ is less than 1.0 (taken from [2])

	Factor	SLS	Reliability class ULS		
		β ≥ 2.8	RC1 β ≥ 3.5	RC2 β ≥ 4.0	RC3 β ≥ 4.6
Model factor	γ_M	1.40	1.40	1.40	1.40
Traffic load p	$\gamma_{f;p}$	1.00	1.05	1.10	1.20
Tangent of internal friction, tan φ′	$\gamma_{m;\varphi}$	1.00	1.05	1.10	1.15
Unit weight fill, γ	$\gamma_{m;\gamma}$	1.00	0.95	0.90	0.85
Subgrade reaction of subsoil, k_s	$\gamma_{m;k}$	1.00	1.30	1.30	1.30
Axial GR stiffness, J	$\gamma_{m;EA}$	1.00	1.00	1.00	1.00
GR Strength, T_r	$\gamma_{m;T}$	1.00	1.30	1.35	1.45

Furthermore, CUR226 (2016) gives the following limitations for its applicability:

– $H / (s_d - d_{eq}) \geq 0.66$ with H, s_d and d_{eq} explained above;
– $P_{traffic} < p_{embankment\ weight}$ or apply κ-model of Heitz (2006, section 6), see section 5 for $p_{traffic}$;
– $b_{eq}/s_{x,y} \geq 0.15$ with b_{eq} the width of a square pile cap or the equivalent width of a circular one;
– one GR layer: $z \leq 0.15$ m, two GR layers: distance between two layers ≤ 0.20 m with z (m) is the distance between GR and pile cap;
– $2/3 \leq s_x/s_y \leq 3/2$;
– $\varphi'_{fill,cv} \geq 35°$ for the lowest layer with height $h^* = 0{,}66\ (s_d - d_{eq})$. Above that, $\varphi'_{fill,cv} \geq 30°$;
– $T_{r,d} \geq 30$ kPa, in both directions, and $0.1 \leq T_{r;x;d}/T_{r;y;d} \leq 10$ where $T_{r,d}$ (kN/m) is the short term GR tensile strength;
– $k_{s;paal}/k_{s;subsoil} > 10$, with k the subgrade reaction.

TRAFFIC LOAD

The traffic loads given in load model BM 1 [5] were included in the CUR226 (2016, [2]). These loads were converted into a uniformly distributed load, resulting in tables with values that were determined as follows:

– the axle loads were spread according Boussinesq over the total height of the embankment;
– the influence of all wheel loads were summed;
– determination of the average stress $p_{traffic}$ on the maximum loaded pile grid $(s_x \cdot s_y)$, with $s_{x,y}$ (m) the ctc-pile spacing.

Table 2 presents a summary of a larger CUR226 (2016, [2]) table, which gives more tables for smaller values for N and for the situation with only one driving lane. When using these tables, the extra spreading capacity of the asphalt top layer may be taken into account with a virtual extra height.

Table 2. Maximum average uniformly distributed traffic load $p_{traffic}$, based on [5] for number of passages per year: N = 2.000.000, 2 driving lanes, with driving lane 1 heavy traffic: 4 wheels F_{wheel} = 120 kN and $q_{uniform}$ = 7.2 kPa and the second driving lane: 4 wheels with F_{wheel} = 100 kN and $q_{uniform}$ = 2.5 kPa (taken from [2])

Height embankment H [m]	Pile spacing			
	1.0 · 1.0 m²	1.5 · 1.5 m²	2.0 · 2.0 m²	2.5 · 2.5 m²
1.0	74.99	70.66	62.11	52.78
2.0	44.04	41.94	39.43	36.77
3.0	28.80	28.01	27.04	25.94

HEAVY TRAFFIC, THIN EMBANKMENT

Heavy truck passages influence the arching in the embankment, specifically in a shallow embankment. Van Eekelen et al. [12] showed that the arching is reduced as a result of a heavy passage. This results in a temporarily increase of the vertical load on the GR. They also showed that arching recovered during a rest period after a number of passages. Heitz [7] conducted experiments with high dynamic loading on a test set-up with four small square piles underneath a sand embankment with and without a geosynthetic basal reinforcement. He found that (1) the arching reduces due to dynamic loading; (2) the arching recovers during a rest period and (3) cyclic loading has significantly less influence on the arching in (*a*) a relatively thick embankment or (*b*) an embankment with GR in comparison to one without GR.

On the basis of his unreinforced experiments, [7] determined an empirical model to reduce arching, the so-called κ (kappa)-model. This κ-model is at the safe side as Heitz based his model on his unreinforced experiments, which are the tests with the heaviest influence on the arching.

The Heitz-κ-model had already been included in CUR226 (2010, [1]). However, the graphs determining the κ-values were modified and brought in line with the original ideas of Heitz.

PILE DESIGN

CUR226 (2010, [1]) assumed that an embankment is not able to re-distribute the load in the case of pile failure (a non-stiff construction). In CUR226 (2016, [2]), this rule has been extended. If the embankment is high enough: $H/(2 \cdot s_d - d_{eq}) \geq 0.66$, the construction may be considered as stiff and the embankment is assumed to be able to re-distribution the load in the case of pile failure.

When the reinforced embankment is non-stiff, which will often be the case for shallow embankments where the pile spacing is maximised, the design should be done using the factors appropriate for a non-stiff structure, unless it has been demonstrated that the structure will continue to perform if one pile fails.

The other calculation rules for the geotechnical bearing capacity of the piles follow the normal local design guidelines ([6] in the Netherlands). Pile moments and cross forces must be calculated with a numerical program using for example finite element analysis, which is further explained in Section 9.

PILE CAP DESIGN

Pile caps are circular or square and preferably have rounded edges to prevent GR damage. If pile caps have sharp edges, protection measures are recommended to protect the GR against damage. A square pile cap size frequently applied in the Netherlands is 0.75 × 0.75 × 0.30 m; which is small and thick in comparison to ‘normal’ concrete structures, this height is usually enough to span several meters. CUR226 (2016, [2]) specifies the loads on the pile caps as follows (Fig. 4):

– vertical load (*A*) due to arching (Fig. 2c), uniformly distributed on the pile cap (kPa);
– tensile force T from the geosynthetic reinforcement (kN/m);
– axial pile force from underneath, which is assumed to be uniformly distributed (kPa).

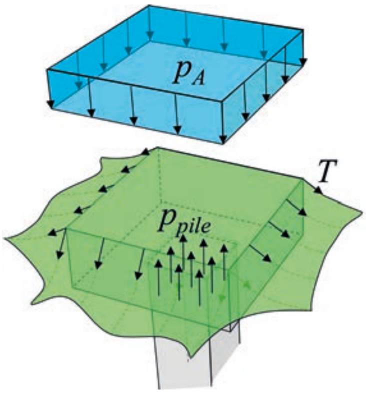

Fig. 4. Loads on a pile cap: arching p_A (kPa), the tensile force T (kN/m) and the axial pile force p_{pile}, uniformly distributed (kPa)

The pile cap should be checked on punching and bending. Usually, the thickness of the pile cap is enough to prevent punching. The strength of the steel reinforcement in the pile cap needs to be enough in the ULS and the sustainability, or the crack width, should be enough in the SLS. The English version of CUR226 (2016, [2]) gives a summary of the more lengthy Dutch pile caps chapter.

NUMERICAL CALCULATIONS

The GR design should be carried out analytically with the Concentric Arches model. CUR226 (2016, [2]) does not allow numerical GR design. However, numerical calculations are usually necessary to determine deformations, pile moments and cross forces. With numerical calculations, the influence is determined:

– on adjacent objects;

– of adjacent existing and future objects;

– of lateral loads such as traffic, spreading forces in the embankment.

In daily practice, 2D calculations are generally used, both in longitudinal and transverse direction. The 3D appearance of piled embankments makes it necessary to consider carefully the pile stiffness, the soil behaviour between piles, pile settlement behaviour and vector summing of pile moments and cross forces.

All relevant construction stages and secondary effects need to be included. A 'gap' needs to be applied between subsoil and GR in the cases that the subsoil support will disappear during service life.

For each cross section, two numerical calculations are needed. The first is conducted with calculation values, although it is an option to use calculation values in the normative phase only, and characteristic values in the other phases. The second calculation must be conducted with characteristic values. The results of the second calculation should be multiplied by 1.2 and should then be compared with the results of the first calculation. The highest values are the normative pile moments and cross forces.

CONCLUSIONS

This paper presents the 2016-update of the Dutch design guideline for basal reinforced piled embankments. This guideline has been developed in full compliance with the Eurocodes, including Eurocode 7 ([6])with its national appendices. The guideline has been published both in Dutch and English.

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