Rigid Inclusion design recommendations on Critical Height for embankment and fill

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ABSTRACT
Rigid Inclusions (RI) or Controlled Modulus Columns (CMC) is a soil reinforcement technique increasingly used in the Canadian market. The rigid inclusion system is constituted of cementitious columns with a diameter ranging from 250 mm to 450 mm installed at a regular pattern. On top of the system, between the top of the RI and the structure to be supported, a compacted layer of granular material called Load Transfer Platform (LTP) is generally installed. Rigid inclusions are often used to reduce settlement induced by embankments. However, when RI are used to support a low height embankment, the thickness of the LTP may be insufficient to fully engage the arching effect, thereby, creating a punching shear failure within the LTP. Some recommendations, based on the principles of limit states, have been developed by the British Standard Institution (BS8006-1), the Federal Highway Administration (FHWA) and by ASIRI on the minimum thickness of LTP ($H_{critical}$). Recommendations are limited and do not distinguish between stiffer and softer subsoil profiles. This article covers the studies and recommendations of Menard, developed from numerical models and case studies, for the design of rigid inclusions under low height embankments.

RÉSUMÉ
Les Inclusions Rigides (IR) ou Colonnes à Module Contrôlé (CMC) sont une technique de renforcement des sols de plus en plus utilisée dans le marché canadien. Le système d'inclusions rigides est constitué de colonnes cimentaires de diamètre allant de 250 mm à 450 mm, installées selon un patron régulier. Au-dessus du système, entre la tête des IR et la structure a supporter, une couche compactée de matériau granulaire appelée plateforme de transfert de charge (PTC) est généralement installée. Les IR sont souvent utilisés pour réduire les tassements induits par des remblais. Cependant, lorsque des inclusions rigides sont utilisées pour soutenir un remblai de faible hauteur, l'épaisseur de la PTC granulaire peut être insuffisante pour engager pleinement l'effet de voûte, créant ainsi une rupture par poinçonnement au sein de la PTC. Quelques recommandations, basées sur les principes d'états limites, ont été développées par la British Standards Institution (BSI), la Federal Highway Administration (FHWA) et par ASIRI sur l'épaisseur minimale de PTC ($H_{critique}$). Les recommandations sont limitées et ne font pas de distinction entre les profils de sol plus raides et les profils plus mous. Cet article couvre les études et recommandations de Menard, développées à partir de modèles numériques et des cas d'études, pour la conception des inclusions rigides sous des remblais de faible hauteur.

1 INTRODUCTION

1.1 Context

Rigid Inclusion (RI) is a recent Ground Improvement technic in the Canadian market. The first rigid inclusion project was done in 2007 by Menard in the province of Quebec. Since then, the technic grew in popularity and has been applied to a diverse range of projects (Residential Building, Warehouses, Wind turbine foundation, Embankment, etc). The RI ground improvement consists of reinforcing the soil with a grid of cementitious columns typically spaced anywhere from 1.5 m to 3.0 m center to center with diameter ranging from 250 mm to 450 mm. Overlaying the grid, a compacted layer of granular material called the Load Transfer Platform (LTP), is generally installed. The load is mostly shared between the soil and the column due to an arching effect in the LTP and partly by friction in the soil (depending on the subsoil stratigraphy).

RI has been proven to be very efficient to reduce total settlement, to be quick to install and to have a greener footprint than the traditional solution: pile and structural Slab or excavation/replacement.

This solution helps to reduce the total settlement and increase the global stability of embankments. It has been used with success to support many embankments in Québec (Beauharnois, Turcot and REM), in other Provinces (Ontario, Manitoba, Alberta, British-Colombia) and in many places in the US and Europe.

For low height embankments, the understanding of the arching mechanism is critical since the thickness of the LTP is limited. ASIRI (2012), McGuire et al. (2011) and the BS8006-1 (2010) established a few guidelines on how to evaluate the critical height $H_{critical}$, which is the minimal LTP thickness needed to ensure a full arching effect and a uniform settlement at surface. This paper aimed to give additional recommendations, aligned with the current state of practice based on a parametric finite element model and validation with case studies.

1.2 Load Transfer Platform Mechanism

The arching effect develops in the Load Transfer Platform (LTP) and can be modeled by an overlapping of the Prandtl diagram above a grid of Rigid Inclusion head (ASIRI), as shown in Figure *1*.

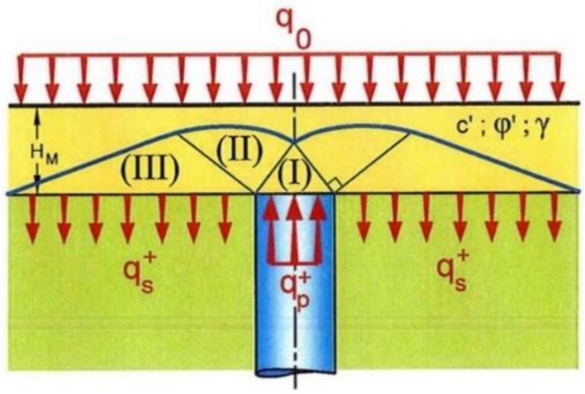

Figure 1 : Prandtl diagram above a rigid inclusion (ASIRI)

When the Prandtl diagram intercepts the LTP surface before it fully develops, additional mechanisms and their effect must be taken into consideration. When a rigid structural element (slab, foundation, mat foundation) is overlaying the LTP, it will induce additional bending moment (Soil-structure Interaction). In case of a flexible structure (road, pavement, fill), differential settlement may occur at surface due to a shear cone opening onto the surface (punching failure mechanism) (Figure 2)

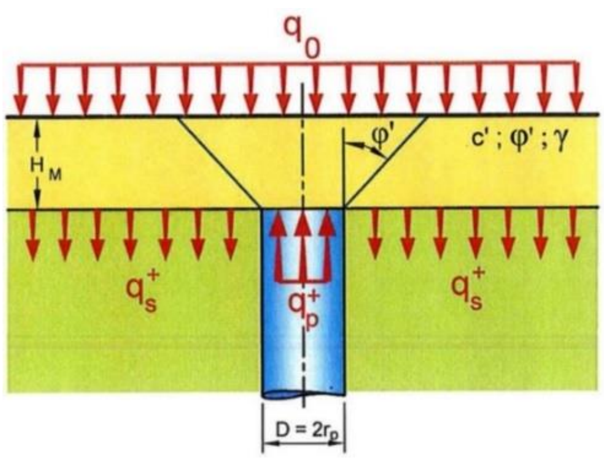

Figure 2 : Shear cone diagram above a rigid inclusion (ASIRI)

For low height embankment (flexible structure), a punching failure will manifest at the surface as an eggbox effect (Figure *3*).

Figure 3 : Example of punching failure of low height embankment over rigid inclusions

A general trend seen with practitioner is to design the LTP with a linear elastic model and validate punching failure analytically. This practice increases the risk on design since a linear elastic model does not include a failure criterion and can often overestimate the load transfer to a RI head and underestimate the differential settlement. A misinterpretation of the failure mechanism will therefore lead into a punching failure of the LTP.

Another trend is to combine the LTP with a geosynthetic. As described by King et al. (2020), the geosynthetic needs a minimum of strain to fully mobilize its tensile strength (typically between 3% and 4%). Ideally, that level of strain would be mobilized during construction and therefore the LTP would be fully active with its maximum tensile strength for the post-construction settlement. This condition is hard to meet and therefore the geosynthetic will not perform as predicted. Also, this assumption does not include the geosynthetic creep over time, which typically range over 1% to 2%, inducing additional settlement over time.

2 CURRENT STANDARD OF PRACTICE

Practitioners can rely on different guidelines to define the LTP thickness (ASIRI/BS8006-1 EBGEO, CUR, Sloan, McGuire). However, there is not yet any agreement on the value of $H_{critical}$, and it can vary significantly from one guideline to another. Typically, from 0.7(s-a) based on the BS8006-1/ASIRI to 1.8(s-a) based on the experimental studies of Sloan et al. (2013). The geometric parameters are defined in Figure *4*. Some recommendations are based on a limit equilibrium methodology (ULS) whereas others are based on full-scale experiments. None of them considers the soil stiffness, which is the key parameter. The mobilization of the arching effect is indeed strain dependent (King et al. 2016) and not considering the subsoil stiffness can lead to both conservative and non-conservative design for serviceability limit state where the differential settlement is of main interest.

The recommendations and analysis are based on the following geometric parameters:

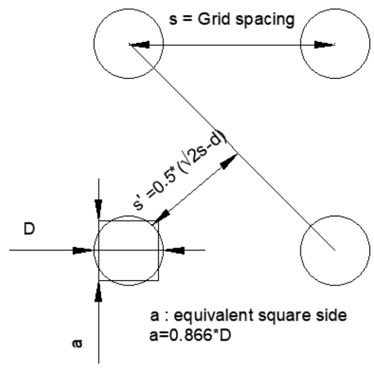

Figure 4 : Geometric parameters

Table 1 : Existing Design Recommendations Summary

	BS 8006-1:2010	McGuire (2011)	Sloan and al. (2013)
Critical Embankment Height $H_{critical}$ *(m)*	0.7(s-a)	0.813s+0.97a w/o traffic loading x1.2 with traffic loading	1.5(s-a) w/o traffic loading 1.8(s-a) with traffic loading
Max. Centre-to-centre CMC spacing *s (m)*	a+1.4H	1.23H-1.19a w/o traffic loading 1.02H-1.19a with traffic loading	a+0.67H w/o traffic loading a+0.56H with traffic loading
Remarks		Bench-scale (1:10 to 1:20) testing, supported by 18 experimental studies and 25 case histories	Experimental study with dissolved geofoam Traffic taken into account during experiments

3 NUMERICAL MODEL METHODOLOGY

To demonstrate which parameters influence the value of $H_{critical}$, a numerical model and a parametric study were developed.

The parametric study presented hereafter is based on the simulation of a large laboratory test called the Trapdoor test. The traditional trapdoor test is conducted by gradually lowering an intact rigid plate between the RI to simulate a progressive reduction of the soil reaction till the failure of the LTP.

To simulate the results of this test, an axisymmetric model centred onto a RI was used. The equivalent radius of the model (R_{model}) is a function of the columns grid spacing (s). The load transfer platform above the columns with a thickness (H) has an elastoplastic behavior with the failure criterion of Mohr-Coulomb. The rigid plate lowering in the laboratory test is simulated with a line displacement at the base of the model. During the simulation, the value of the prescribed settlement will increase step after step from 0.2 mm (typical of a stiff soil) to 600 mm (typical of a very soft soil). The rigid inclusion head is modeled by fixed boundary conditions, along the RI radius.

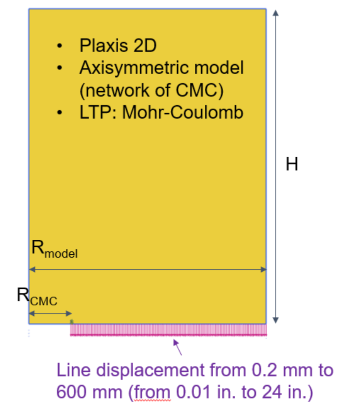

Figure 5 : Finite Element Model geometry

The parametric range of this study is shown in Table *2*.

Table 2 : Model Parameters Boundary

Parameters	Minimum boundary	Maximum boundary	Remarks
RI diameter (D)	280 mm (11 in.)	400 mm (16 in.)	Typical range
Spacing (s)	1.5 m (5 ft.)	3 m (10 ft.)	Typical range
Incorporation ratio	0.7%	5%	Typical range of RI ground improvement
s-a	1.2 m (4 ft.)	2.7 m (9 ft.)	Large range value
Friction angle (°)	30	40	Typical range for granular LTP

For each value of prescribed displacement, for each value of soil stiffness, this model evaluates both the load transfer between the RI qp+ and the soil qs+ and the differential settlement at surface.

The figure 5 presents, for one geometry, the stress in the surrounding soil qs+ at the RI top level and qp+ the stress at the column head. The blue line represents the load conservation equation of the system. If the stress at the column head qp+ is zero, then 100% of the load passed onto the soil. This first scenario would represent a very stiff soil, where the RI contribution is minimal.

The initial prescribed displacement is 0.2 mm. It corresponds to stiff soil with a minimal transfer to the RI head. Progressively step after step, the prescribed settlement is increased, corresponding to a progressive decrease of the soil reaction, hence the stress onto the column increases. At a certain point even if the displacement is increased, the load at the columns did not change. The failure of the load transfer platform has occurred.

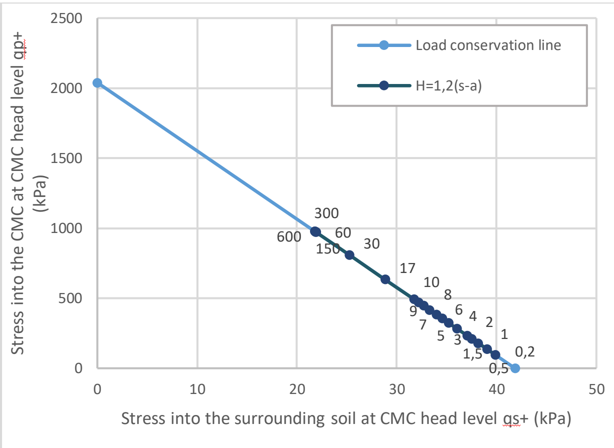

Figure 6 : Incremental load transfer based on deformation.

In figure 6, the results of the different geometries in terms of H/(s-a) are plotted. In the right side, the failure of the LTP (the upleft edge of each colored line) can be linked by a straight line (dash line in red). The equation of this line is equal to the failure mechanism equation proposed by Prandtl (ref ASIRI and figure 1).

On the left side, the dashed line in red connects the failure points following to a curve which matches with the shear cone / punching failure mechanism (ref ASIRI and figure 2). At a certain thickness of the LTP named $H_{critical}$, the failure of the LTP passes from the Prandtl mechanism to a punching mechanism where a high differential settlement is observed at the top of the LTP (embankment).

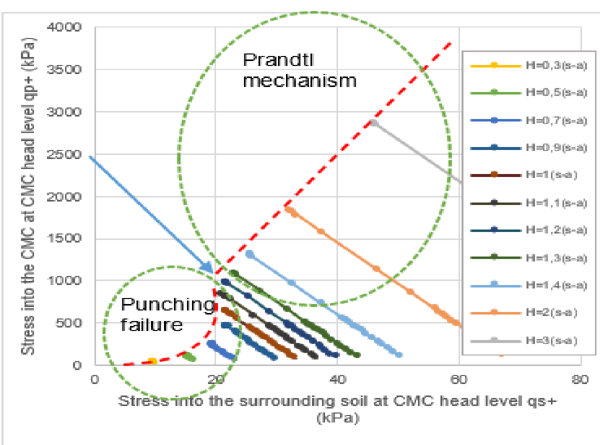

Figure 7 : Load Conservation line in a LTP

For each studied cases, the evolution of the differential settlement at the surface according to the geometry ratio H/(s-a) for different values of prescribed displacements / soil stiffnesses. A prescribed displacement of 0.2 mm represents a stiff soil whereas a prescribed displacement of 600 mm represents a very soft soil.

This analysis shows that the soil stiffness has a direct impact on the $H_{critical}$ value, which is the minimal LTP thickness ensuring a flat vertical displacement at surface.

If the prescribed displacement is lower than 4 mm (Figure *8*), characterising relatively stiff soils, $H_{critical}$ appears to be equal to 0.7(s-a) which corresponds to the BSI recommendation.

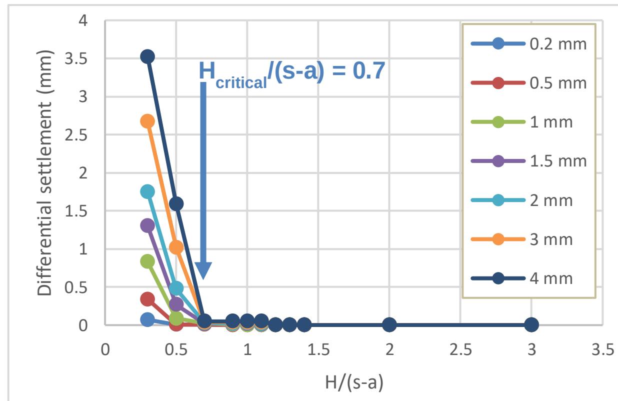

Figure 8 : Differential settlement according to the geometry ratio H/(s-a) for small prescribed displacement

If the prescribed displacement is between 4 mm to 600 mm (Figure *9*), characterising very soft soils, $H_{critical}$ appears to be higher than 1.2 (s-a), getting closer to Sloan and McGuire recommendations.

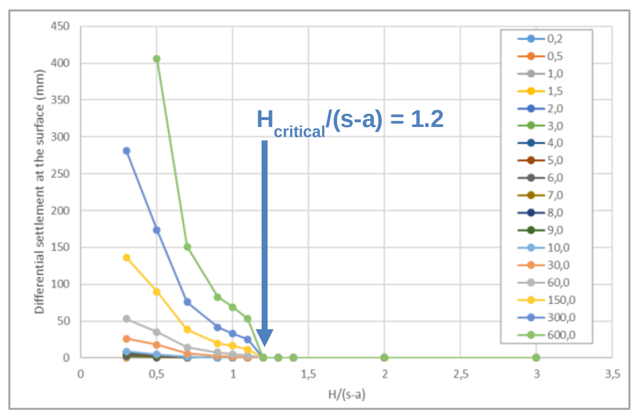

Figure 9 : Differential settlement according to the geometry ratio H/(s-a) for large prescribed displacement

This numerical analysis presents some inherent limitations. First, it does not account for any progressive decompaction of the LTP that can be observed due to the soil settlement and the dynamic effects of the traffic loading. These aspects are difficult to numerically capture. Then, the influence of the LTP friction angles is not detailed in the present paper. These results are qualitatively valid for granular LTP with friction angle varying from 30° to 40°. Higher the friction angle is, lower are the risks of LTP failure and/or differential settlement at the top of the embankment.

4 DISCUSSION AND RECOMMENDATIONS

Based on the extensive numerical analysis, Menard has established the following internal recommendation in Table *3*, valid for low height embankment made of a granular LTP material, well compacted with a friction angle minimum of 34°, and not covered by a rigid structure.

Table 3 : Menard Recommendation for RI design for low height embankment

Expected settlement (w) including creep **without ground improvement**	Menard Recommendations
w < 5 cm (2 in.)	No recommendation
5 cm (2 in.) ≤ w ≤ 12.5 cm (5 in.)	$H_{critical} = 0.7(s-a)$ $\boldsymbol{s \leq a + H/0.7}$
12.5 cm (5 in.) ≤ w ≤ 25 cm (10 in.)	$H_{critical} = 1.0(s-a)$ $\boldsymbol{s \leq a + H}$
25 cm (10 in.) ≤ w ≤ 50 cm (20 in.)	$H_{critical} = 1.5(s-a)$ $\boldsymbol{s \leq a + H/1.5}$
w > 50 cm (20 in.)	$H_{critical} = 1.8(s-a)$ $\boldsymbol{s \leq a + H/1.8}$

The main advantage of those recommendations is to have a gradual evaluation of $H_{critical}$ based on the subsoil condition. The development of the arching effect is strain-dependent and therefore the possibility of reaching failure is also strain-dependent.

The ASIRI/BSI recommendation is adequate when the settlement potential is low and therefore the potential of shear punching failure is low. Also, for stiff subsoil layer, an arching effect within the reinforced layer is highly probable and will help mitigate any punching.

However, as underlined by King et al (2020), the Asiri/BSI guideline was developed based on cased studies involving firm clay. For soft or very soft soil, ASIRI/BS8006-1 recommendation is not appropriate and might lead to failure at both serviceability and ULS.

By evaluating the total settlement (including the creep settlement) without RI and adjusting $H_{critical}$ accordingly, those recommendations ensure the design to be on the safe side.

Nine case studies were compared to evaluate the performance of the recommendation (see Appendix A). The case studies come from literature and/or from private data (Menard and others). Two (Sites 2 et 9) of the nine case studies showed differential settlement at the surface. ANNEXE presents the data of those projects.

Figure *10* compares Menard's recommendations with the other guidelines and various case studies in a normalized geometrical space (H/d and s'/d). The definition of s' and the other are in Figure *4*. Since this is the same geometrical space used in King et al (2017), the data from King et al (2017) were also plotted.

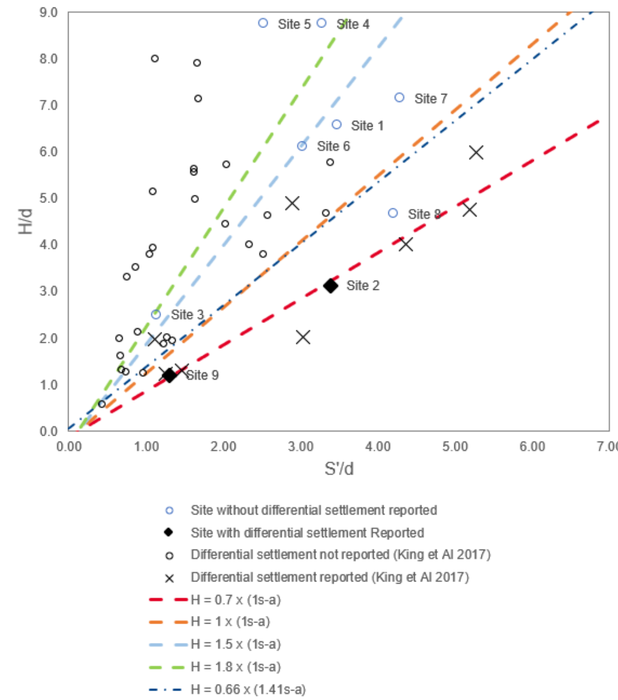

Figure 10 : Case Studies comparison with Menard Recommendation in Normalize geometrical parameters.

The recommendations are in good agreement with the other recommendations. If the initial settlement without RI is low, the recommendation is located between the BS8006-1 line and the critical height line developed by McGuire. For large settlements, Menard's recommendation is more stringent than the line proposed by McGuire.

Case studies were plotted in Figure *10* to evaluate the performance of our recommendations. Three projects (Site 9, Site 2, Site 8) are located on the BS8006-1 limit characterized by $H_{critical}$ = 0.7(s-a). This limit is the least conservative of those existing today.

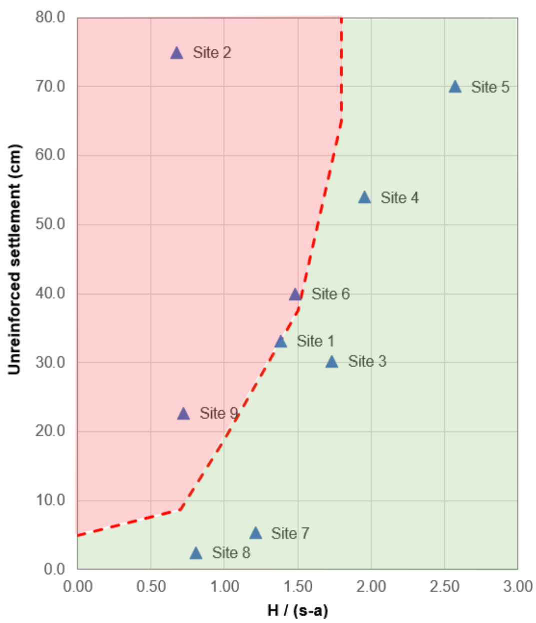

Figure 11 : Case studies compared to Menard recommendation depending on unreinforced settlement

Among these three projects, only two presented unacceptable deformations and required repair works. One might think that the lower the s'/d ratio (i.e. the closer together or the larger the diameter of the columns), the safer it is. Following this reasoning, we would be tempted to say that Site 8 is the one presenting the most critical configuration of the three. And yet, it is the only one of the 3 that did not present any unacceptable deformation.

If we compare the same projects but in Menard recommendation space (unreinforced settlement vs normalized geometry H/(s-a): Figure *11*), it is possible to see that Site 8 has a settlement of less than 5 cm while Site 2 and Site 9 have settlements respectively higher than 20 cm and 70 cm. Based on Menard recommendations, Site 2 and Site 9 projects are widely outside the allowable domain as opposed to Site 8 who is within the allowable domain. Since the data from King et al (2017) did not provide the unreinforced settlement, they were not used in Figure *10*.

The other projects that performed well are in good agreement with the recommendations. The projects in Site 1 and Site 6 are slightly outside the allowable domain. Their performance indicated that there is a certain level of uncertainty with the definition of the limits. However, based on the available data, all projects within the allowable domain performed well, indicating the recommendations are on the safe side.

5 CONCLUSION

Menard recommendations have been developed based on a series of numerical analysis and compared with case studies. The recommendations shown to be on the safe side of design. By adding the notion of unreinforced settlement and adjusting the $H_{critical}$ based on this parameter, those recommendations aimed to be more practical and to avoid using stringent criteria on stiff soil profile. However, for very soft soils, practitioners should be more cautious and use stricter criteria than the one recommended by BSI (BS8006-1)/ASIRI and even McGuire if large creep settlements are involved.

The parametric study and recommendations have some limitations:

- This study assumed that no arching effect happened below the RI head. The presence of compacted granular material or very stiff clay, like a clay crust, between the RI head and soft layers might reduce the failure potential.
- Those recommendations also exclude the impact of geogrid in the LTP. As described in section1.1, the strain mobilization of geosynthetic is generally larger than the strain range of the arching effect. Some experiments show that geosynthetics can increase load transfer, but only if they are put in tension before the arching effect forms in the LTP. As the geosynthetic effect requires a higher differential settlement than that needed to activate the arch mechanisms, it must be activated quickly during the construction phase. However, this is rarely the case and most of the load is transferred by arching effect. It is the authors opinion that the RI designer should prioritize smaller diameter with a higher redundancy over a large diameter/spacing with use of geosynthetic.
- Finally, this study assumed a uniform LTP material over the range of $H_{critical}$. For many low-height embankments, the road sub-foundation is composed of many materials with different levels of friction angle, cohesion and, in some cases, cementations. The effect of composite LTP material on load transfer mechanism and failure criteria is not well understood and documented. Additional studies on this subject would be beneficial for the RI design for low-height embankments.

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7 ANNEXE

Project	D_C (m)	H (m)	$s_{square\ grid}$ (m)	a (m)	$H/(s-a)$ (-)	$u_{z,w/o\ SR}$ (cm)	d (m)	s' (m)	s'/d (-)	H/d (-)
Site 1	0.32	2.1	1.80	0.284	1.38	33.2	0.32	1.11	3.48	6.56
Site 2	0.4	1.25	2.20	0.354	0.68	75.0	0.4	1.36	3.39	3.13
Site 3	0.8	2	1.86	0.709	1.74	30.2	0.8	0.92	1.15	2.50
Site 4	0.28	2.45	1.50	0.248	1.96	54.0	0.28	0.92	3.29	8.75
Site 5	0.28	2.45	1.20	0.248	2.57	70.0	0.28	0.71	2.53	8.75
Site 6	0.36	2.2	1.80	0.319	1.49	40.0	0.36	1.09	3.04	6.11
Site 7	0.28	2	1.90	0.248	1.21	5.3	0.28	1.20	4.30	7.14
Site 8	0.3	1.4	2.00	0.266	0.81	2.5	0.3	1.26	4.21	4.67
Site 9	0.91	1.1	2.33	0.806	0.72	22.7	0.91	1.19	1.31	1.21